INSTITUTE OF HYDROLOGY

MAHAWELI GANGA DEVELOPMENT

VICTORIA PROJECT

REPORT
ON THE
HYDROLOGICAL ANALYSIS

This report is prepared for

Sir Alexander Gibb & Partners, Reading

by

Institute of Hydrology
Wallingford
Oxon
UK

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INTRODUCTION

The Victoria Multipurpose Project is one of the component schemes of the complex system of dams, hydroelectric plants, and irrigation canals that make up the Mahaweli Ganga Development Project. Out of Phase I of the overall development plan proposed by the UNDP/FAO, only the first project, the diversion barrage and associated works at Polgolla, has so far been completed. In the original plan, as well as the current Accelerated Programme, the Victoria Project will be the second scheme to be implemented.

The Project involves the construction of a dam in the deep valley of the Mahaweli Ganga immediately upstream of the rapids at Victoria Falls (Figure 1). Regulated flows from the dam will be discharged into the main river and then diverted into a canal on the right bank by a new anicut to be constructed at Minipe, about 25 km downstream. This canal picks up flows from several tributaries on the right bank before feeding into the Ulhitiya Oya, where another dam is to be constructed. Other schemes in future phases of the Development Project involve the construction of major structures on the right bank tributaries; these plans have not been considered in the present study.

The proposed dam at Randenigala, a few kilometres upstream of Minipe, will introduce additional regulation downstream of Victoria. This proposed scheme, and the existing works at Polgolla will affect the Victoria Project directly and are therefore considered here.

The main objectives of this hydrological study are to provide consistent sets of discharge and climatic data as inputs to a simulation model of the reservoir system, and to provide flood estimates for engineering design.

GAUGING STATIONS

GAUGING STATIONS

1 Peradeniya
2 Gurudeniya
3 Victoria
4 Randenigala
5 Weragamtota
6 Teldeniya
7 Talawakanda
8 Ulhitiya
9 Maduru Oya
10 Gallodai Aru

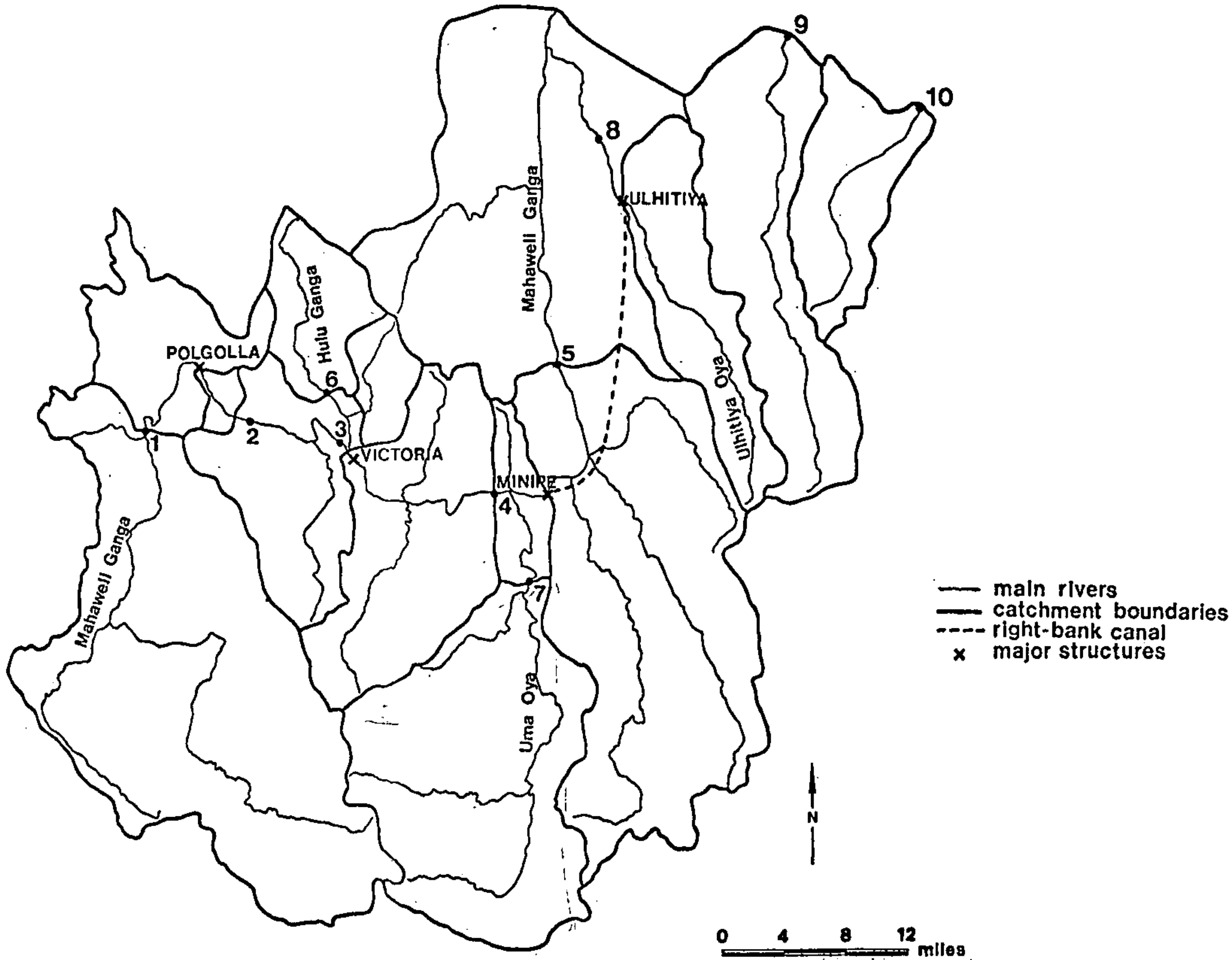


Figure 1

Climate

Sri Lanka has a tropical climate, in which the pattern and distribution of rainfall are determined largely by the south-west and north-east monsoons that occur in the periods May to September and December to February respectively. Of lesser importance is the rainfall of the inter-monsoon periods that can be caused by either convective or cyclonic activity.

The distribution of rainfall during the monsoons is determined by topography, with large falls occurring on windward slopes and smaller falls in the rainshadow on leeward slopes. The Central Highlands are the dominant orographic feature, and separate the areas of high and low rainfall that alternate from the west to the east with each monsoon. They also form the boundary between the 'wet zone' to the west, where rainfall is in surplus throughout the year, and the 'dry zone' to the north-east where rainfall is more seasonal.

A significant part of the headwaters of the Mahaweli Ganga is in the wet zone where the rainfall is a result of both the south-west and north-east monsoons. Rainfall in this part of the catchment is high and its seasonal distribution is determined by the timing of each monsoon. Consequently, the main drainage network is assured of a large, though fluctuating, perennial flow.

In contrast, the area to the east of the highlands receives rainfall during the north-east monsoon and is scarcely affected by the south-west monsoon. The runoff from the catchments in this area is therefore highly seasonal with a distinct dry period from May to September.

The study area

The area considered for the present study includes the main catchment of the Mahaweli Ganga down to the gauging station at Weragamtota and the right bank tributary of the Ulhitiya Oya, covering a total area of over 2500 km^2. The total catchment area of the river to the sea is over 10400 km^2.

The river rises in the Central Highlands, where the highest peak is over 2500 m, and falls over its length of 320 km to sea level at Trincomalee. The geology of the upper catchment is characterised by metamorphic rocks, notably charnokite and quartzite; here the terrain is very steep and the soil cover is thin. The river flows for about 160 km through these mountainous regions, where the valleys are often very deep and the bedslopes steep, before reaching the plains near Minipe and turning northwards. By contrast in this lower region, where the main features of the geology are hornblende and gneiss, the terrain and river bedslopes are much flatter, and the river meanders become more pronounced.

AVAILABLE DATA

Rainfall

An extensive raingauge network has been in existence since 1920 and there are many stations with records that extend back to the nineteenth century. There is a particularly dense network of gauges in the upper catchment of the Mahaweli Ganga, where the existence of individual gauges can often be associated with the development of tea estates. However, in the region to the east and north-east of Minipe, where the vegetation is mainly parkland, jungle or scrub, there are few reliable, long-term gauges. Observations at these gauges are usually taken once or twice a day, and summaries of the data are published annually in the Report on the Colombo Observatory (1). Copies of the daily data for each gauge are kept at the Meteorological Department.

A number of recording raingauges have also been installed in the upper catchment by the Meteorological Department. These data are available for a much shorter period than those for the standard gauges mentioned above, and they have not been processed for publication locally. However a recent paper (2) has used the data from these recording raingauges in a rainfall-depth-duration-frequency study for the whole of Sri Lanka. The original hyetographs from the recording gauges at Kandy and Nuwara Eliya were available for inspection at the Meteorological Department.

The long-term mean annual falls at each raingauge were compared with data from nearby gauges. Some apparent inconsistencies arose from uncertain location of individual gauges on the map; the locations of these gauges were checked at the Meteorological Department and the inconsistencies resolved. Any abnormally high or low falls were also verified against the original data.

The monthly rainfalls have been used by the Irrigation Department to calculate the rainfall on catchments where runoff is recorded, using the method of Thiessen polygons. These data are included on the summary sheets of the runoff data, recorded at individual gauging stations, and kept at the Irrigation Department.

Runoff

Annual discharge data have been published (3) for at least 22 gauging stations in the whole Mahaweli Ganga basin. These data are given as annual runoff volumes for a common period from 1944/45 to 1970/71. At some of the stations, where the data are incomplete, the missing records have been estimated by a method of correlation that is not described in detail. Summaries of the monthly discharge data have also been published annually (4).

The locations of the gauges considered in this study are shown in Figure 1, and Table 1 summarises the availability of the data from these stations. Note that as the gauge at Manampitiya is about 70 km downstream of Weragamtota, it is not shown on Figure 1.

Although a continuous water level recorder was installed at Gurudeniya in 1972, it was removed once the barrage at Polgolla was commissioned. With the exception of the short record from this recorder, all the flow data used in this study are derived from hourly measurements of river stage made by local observers at the gauging stations.

At the more important gauging stations, that are invariably located on the Mahaweli Ganga itself, stage readings are entered for each hour on the gauge returns. At the other stations, however, the observations are restricted to the daylight hours alone. Consequently the hydrographs of any floods that occurred during the night may not have been observed. This is particularly important for the tributaries, as their catchments can respond rapidly to local rainfall, and high flows do not normally persist for more than about 24 hours.

TABLE 1

AVAILABILITY OF RUNOFF RECORDS

River	Station	Period of record from	to
Mahaweli Ganga	Peradeniya	1944/5	1974/5
"	Gurudeniya	1944/5	1974/5
"	Victoria	1963/4	1968/9
	"	1973/4	1974/5
"	Randenigala	1954/5	1974/5
"	Weregamtota	1945/6	1974/5
"	Manampitiya	1944/5	1974/5
Hulu Ganga	Teldeniya	1954/5	1974/5
Galmal Oya	Moragamulla	1963/4	1974/5
Maha Oya	Hanguraketa	1954/5	1959/60
Uma Oya	Talawakanda	1957/8	1974/5
Loggal Oya	Meegalakula	1955/6	1964/5
Ulhitiya Oya		1953/4	1959/60
Maduru Oya	Kandegama	1951/2	1956/7
Gallodai Aru		1945/6	1974/5

All the gauging stations on the Mahaweli Ganga shown in Figure 1, as well as some of the tributary stations, were visited. At many of the sites, two staff gauges had been installed: one for measuring low stages and the other for floods. The lower gauge boards are generally embedded in the river bank, and the higher boards attached to a convenient tree or bridge pier. At several sites, the method of fixing the gauge boards appeared to be insecure; at Teldeniya for example we saw that the level of one board with respect to the other was not consistent.

The reaches in which the individual gauging stations are located, are in general reasonably well suited for discharge measurements, although at some sites the river banks are not very well defined. But at Victoria the original gauge was installed on a wide bend of the river, immediately upstream of the confluence with the Hulu Ganga, where there are many rocks and the flow appeared to be extremely turbulent. This gauge was moved about two years ago to a more suitable site about 2 km upsteam. It was clear that on the main river, the shifting sandbanks that can form on top of the river bedrock could cause significant changes in the bed profile at low flows. In other respects the cross sections at the stations visited appeared to be stable, but no sequential topographic surveys had been made to allow this observation to be checked.

At most sites, a cable has been fixed across the river to enable boat gaugings to be carried out more easily, but there is no provision for suspending a current meter from the cable to measure high discharges. Consequently, the only reliable discharge measurements are those made at low flows, for once the river stage begins to rise it then becomes too dangerous to use a boat. At sites where there is no cable, gauging is carried out by wading and discharge measurements are again limited to low-flows. Hilger-Watt current meters are used extensively by the Irrigation Department. They appear to be used with care and to be in good condition. It is, however, unclear how often the calibration of an individual meter is checked.

The frequency of discharge measurements varies from site to site: on the main river, for example, up to three measurements a day are made, whereas on some of the smaller tributaries there may be gaps of many months between gaugings. Rating curves at low stages have been derived at each site from these discharge measurements, and have then been extrapolated to higher stages using a simple log-log relationship. This method does not take account of any discontinuities in the river cross-section with increasing stage.

The gauge return, analysis and hydrological data files were available for inspection at the Irrigation Department, where the basic data processing is also carried out. The mean daily discharge is calculated from the mean daily stage and the appropriate rating curve. When floods occur during the night and are not recorded, estimates of the peak and total discharge are made to estimate the mean daily flows. The mean daily discharges are tabulated for each station, together with summaries of the monthly and annual means and, where available, the annual maximum instantaneous discharge. Estimates of the mean catchment rainfall are also tabulated and are calculated using the Thiessen polygon method.

The relevant information from these data files was copied out by hand, once examination of the analysis and gauge return files had suggested that the records for an individual station could be used with confidence. Simple double mass curve analysis on the monthly records for Peradeniya, Gurudeniya and Randenigala suggested that these data are internally consistent. A similar analysis using Teldeniya and Moragamulla showed discrepancies between the two stations. Moreover for several years, the runoff of Teldeniya, which has the longer record, bears little resemblance to the corresponding catchment rainfall. Subsequent examination of the analysis and gauge return files showed anomalies that led us to discount all but the flood data from this gauge on the Hulu Ganga.

The gauge at Victoria Falls is located immediately above the confluence of the Hulu Ganga and Mahaweli Ganga. The records do not cover a consistently long period and as there are sometimes unexplainable losses along the reach from Gurudeniya to Victoria, they have also been excluded from further analysis.

In view of the limited time available for this study, we have been unable to reprocess any of the basic data. We have therefore made some simple quality control checks, described above, and also made subjective judgements on the quality of the data from each gauging station. Thus, it was decided that the most reliable data for the main gauging stations were from the Mahaweli Ganga, and that the best tributary data were from the Uma Oya at Talawakanda.

As mentioned earlier, the flows in the Mahaweli Ganga are perennial, but with important seasonal fluctuations. These effects are illustrated in Figure 2, where the mean monthly discharges of the recorded data at each of the main river gauging stations are plotted. The incidence of the north-east monsoon is clearly shown by the pronounced peaks at the downstream stations. The variability of the monthly totals is illustrated in Figure 3, which shows the coefficient of variation (defined as the standard deviation divided by the mean) for each month. The high variability at the upstream stations in December and May can be accounted for by changes in the timing of the start of each monsoon from year to year. A similar effect can be seen at the downstream stations for the start and end of the north-east monsoon. The variability of the south-west monsoon flows appears to become progessively damped out downstream.

Evaporation

The data required for calculating evaporation, using Penman's equation, are available from three climatological stations within the study area, the locations of which are shown in Figure 4. The data for each station have been processed by the Meteorological Department and have been summarised as monthly means; these were used in a computer program, described elsewhere (5) to calculate the values of open water evaporation, E_o, given in Table 2. Potential transpiration, E_t, was calculated using an assumed value of 0.25 for the albedo at each station; these results are also given in Table 2.

Several agrometeorological stations have also been set up recently, but only one or two years of data have been recorded so far.

MEAN MONTHLY DISCHARGE ON THE MAHAWELI GANGA

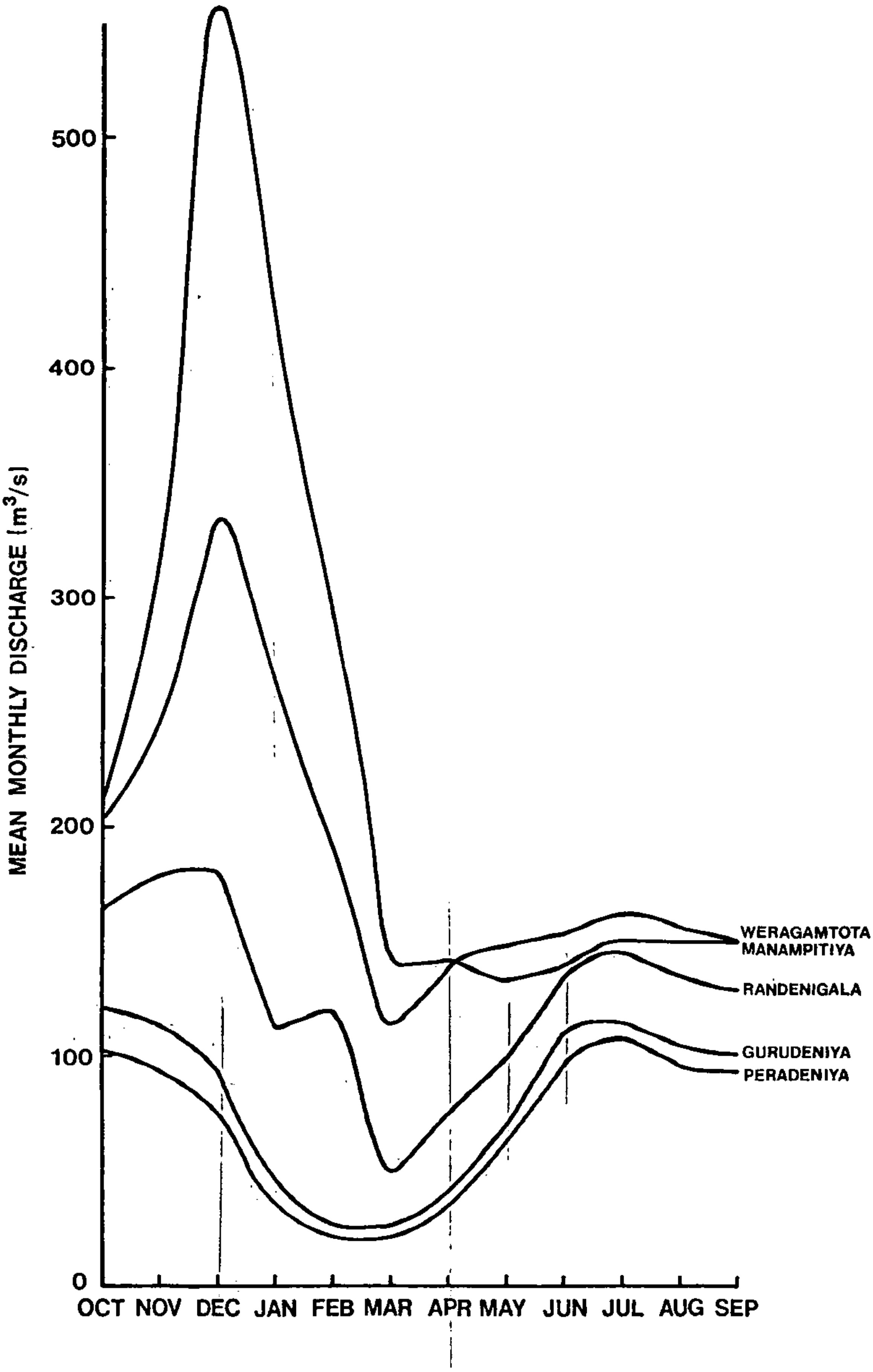


Figure 2

VARIABILITY OF MONTHLY DISCHARGE ON THE MAHAWELI GANGA

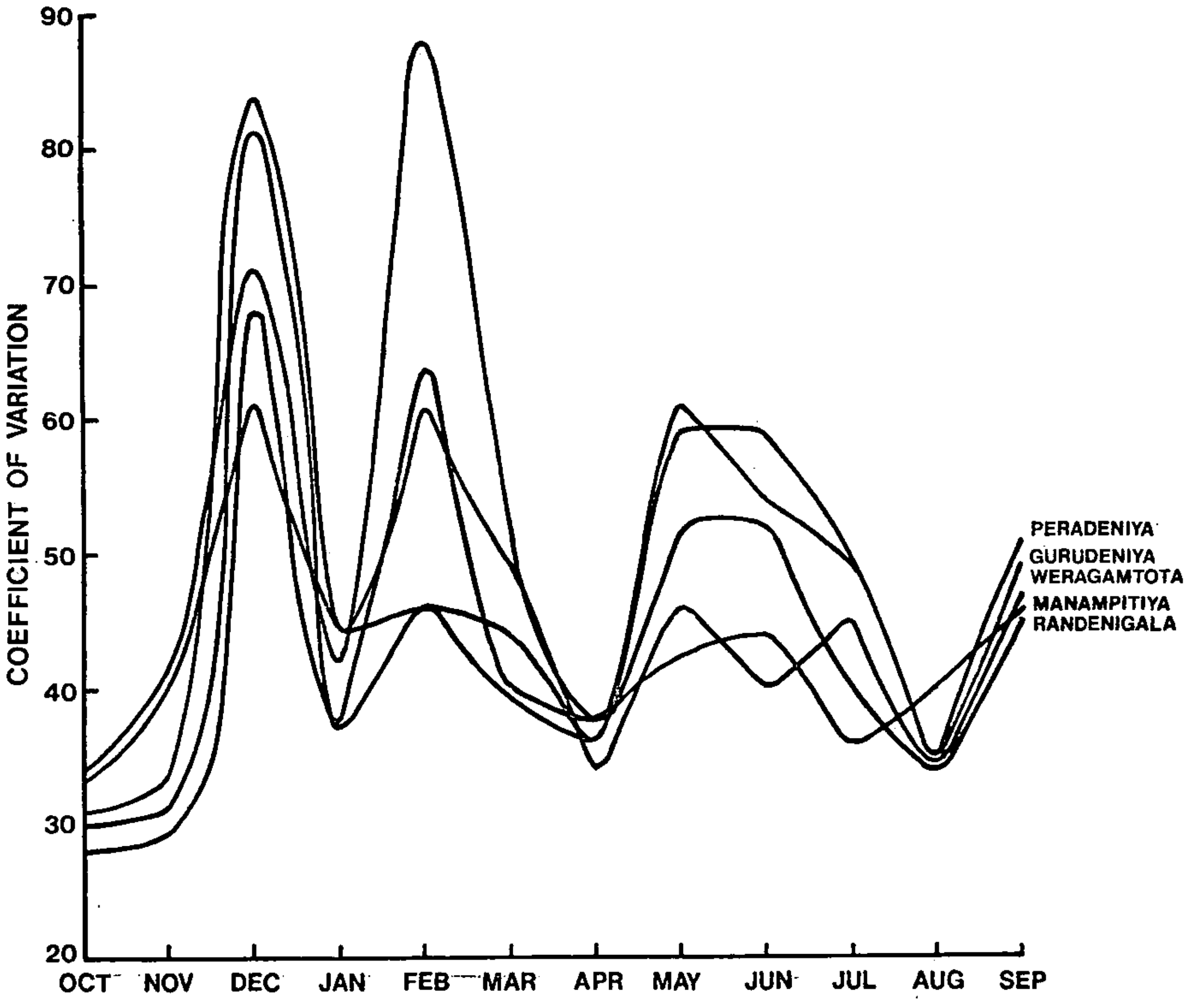


Figure 3

TABLE 2

ESTIMATES OF EVAPORATION
(mm)

Month	O	N	D			M	A	M			A	S	Total
Station													
Open water evaporation, E_o													
Kandy	169	148	151	168	183	223	191	193	164	171	178	181	2121
Diyatalawa	135	106	106	114	122	147	131	147	154	163	165	158	1648
Nuwara Eliya	118	109	109	120	127	156	135	124	111	110	114	115	1447
Potential transpiration, E_t (albedo – 0.25)													
Kandy	133	146	180	151	154	130	136	141	144	133	116	121	1686
Diyatalawa	85	93	113	101	116	125	131	131	123	104	81	80	1284
Nuwara Eliya	91	98	120	103	96	87	87	90	90	90	83	83	1117

Note: Some apparent discrepancies may result from rounding to the nearest millimetre.

RAINFALL & CLIMATOLOGICAL STATIONS

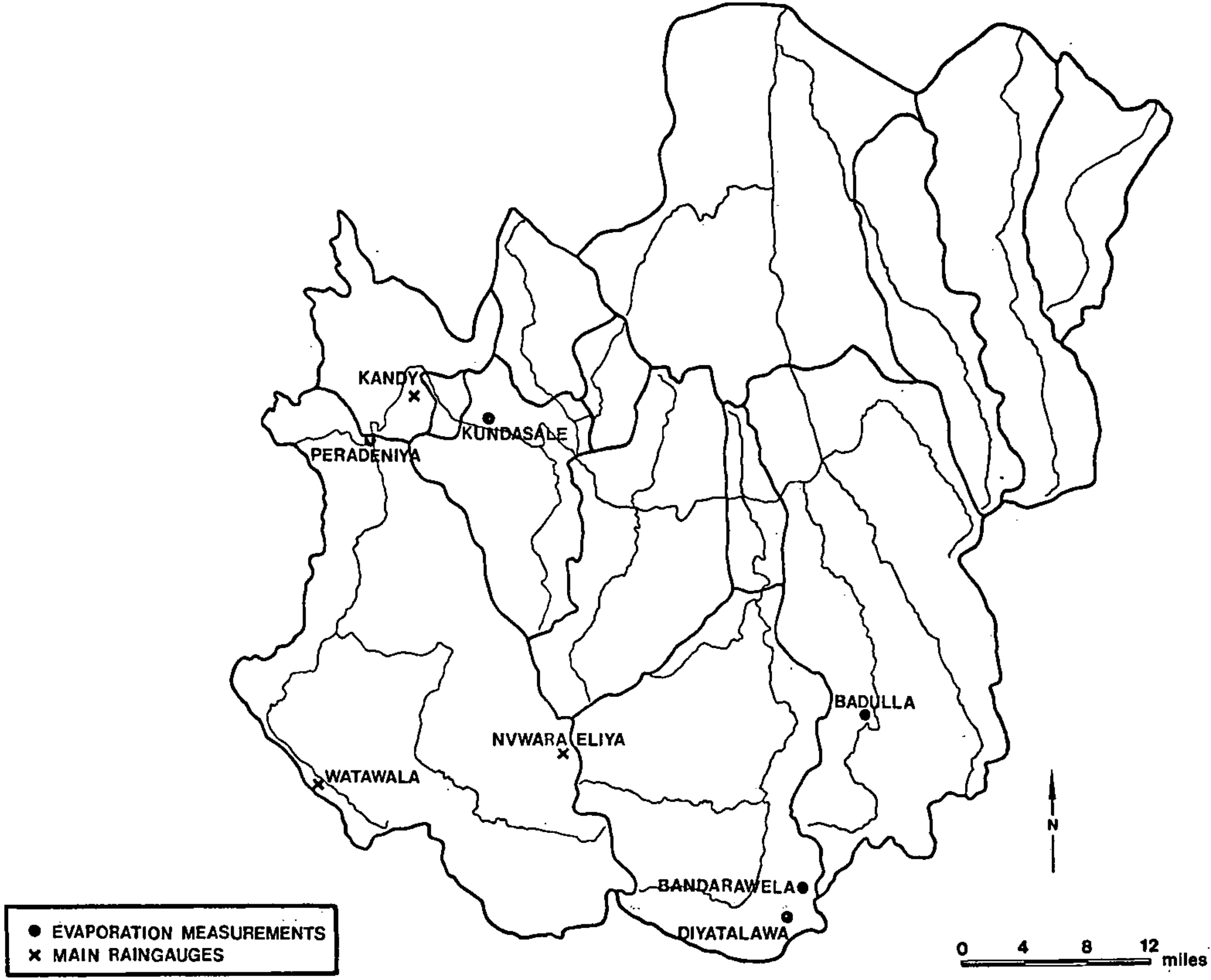


Figure 4

The monthly evaporation calculated at these stations is consistent with the results from the long-term stations, and is included in subsequent analysis.

Sedimentation

Few sediment transport and bedload measurements have been made on the Mahaweli Ganga, so estimates of the loss in reservoir storage must be based on the limited data from Gurudeniya. The data have been used to compile the sediment and bedload rating curves used by SOGREAH (6) and a consistent set of curves are held in the Irrigation Department files.

In the absence of any other data, the sources mentioned above, which use data from 1970 to 1971, 1973 and 1975, have been used here to derive the following relationship:

$$Q_T = 6.5 \times 10^{-5} . Q^{2.03}$$

where Q_T is the total sediment and bedload in tons/day
and Q is the mean daily discharge in ft^3/sec.

WATER RESOURCES

Mean annual rainfall

As the Thiessen polygon method for calculating areal rainfall does not take the local topography into consideration, it is often preferable to use an alternative approach, such as the isohyetal method, in regions where rainfall is determined by orographic effects. Moreover in the upper Mahaweli Ganga catchment, rainfall data are available from a dense network of gauges for the standard period from 1931 to 1960. This isohyetal approach is therefore particularly suited to this analysis.

The position of each raingauge within the study area was plotted on a 1:250 000 topographic map, together with the corresponding value of mean annual rainfall calculated for the standard period (1). The relationship between rainfall and location, topography and aspect was particularly noticeable; the shape of the isohyets drawn in Figure 5 is therefore influenced by the topography between individual gauges. An attempt was made to relate rainfall with elevation, but although these are correlated in parts of the area, the position of a gauge with respect to the local relief is more important than elevation in determining rainfall.

The most conspicuous features of the isohyetal map are the very heavy falls west of the main massif near Nuwara Eliya, and the heavy falls on Knuckles Ridge to the north-east of Kandy. By contrast, to the east of Kandy, there is a band of lower rainfall centred on the valley of the Mahaweli Ganga where it is sheltered from both the south-west and north-east monsoons; the precise extent of this drier area is not well defined because of a shortage of raingauges along the valley, but it has been inferred from the relief. Another area of low rainfall lies in the similarly sheltered valley of the Uma Oya between Badulla and Bandarawela.

In general, the distribution of rainfall is well defined by the available records, but there is a shortage of gauges in some of the low-lying, drier areas. This reinforces the decision to use the

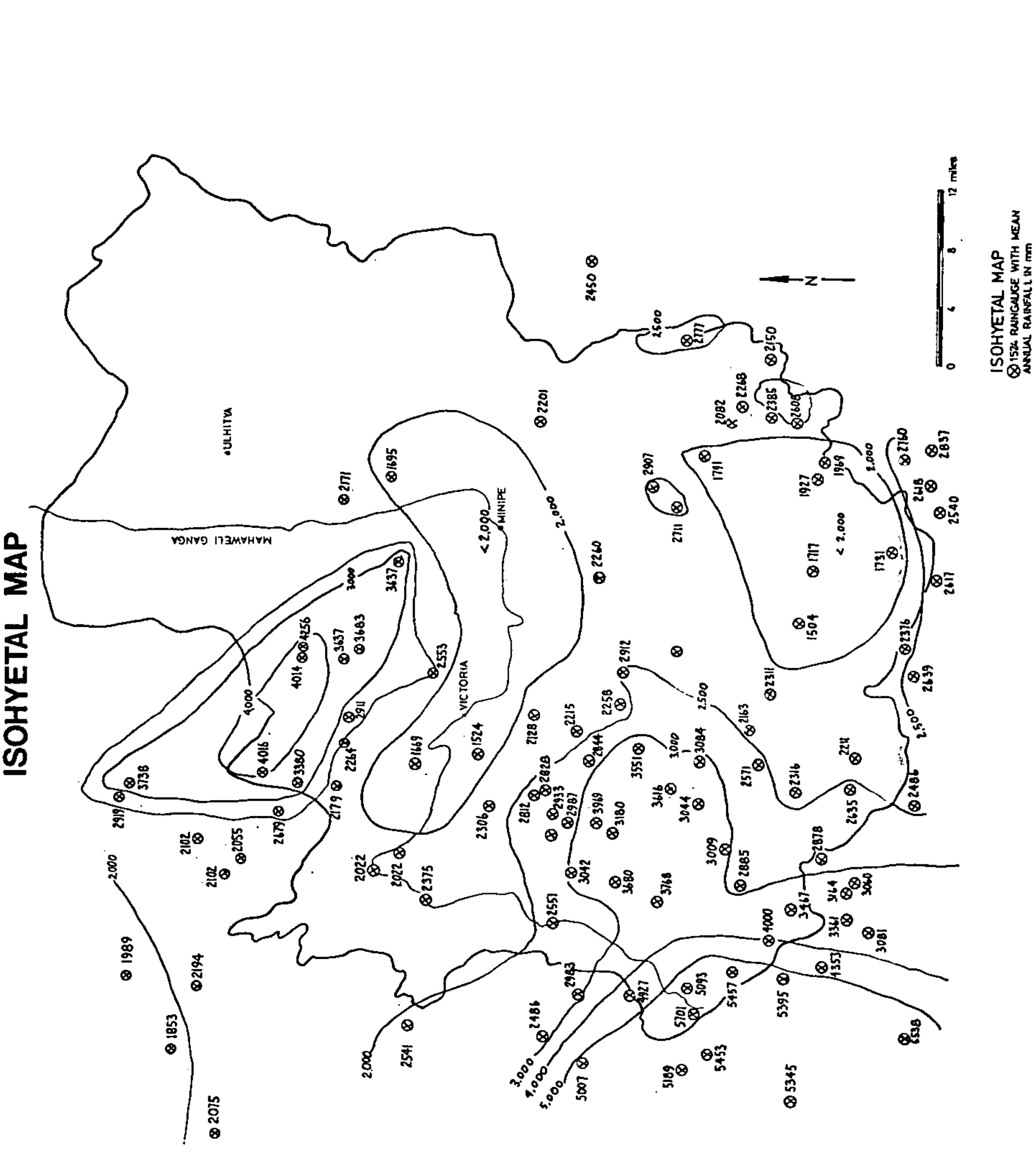

ISOHYETAL MAP
MAHAWELI GANGA
ULHITIYA
MINIPE
VICTORIA
ISOHYETAL MAP
⊗1524 RAINGAUGE WITH MEAN ANNUAL RAINFALL IN mm
0 4 8 12 miles
N

Figure 5

isohyetal map which takes full account of the topography, rather than the Thiessen polygon method. The long-term mean annual rainfall for the project catchments has therefore been calculated from Figure 5.

A second rainfall map (Figure 6) shows the maximum daily falls recorded at each gauge during its period of record and the date of the event. A striking feature of this map is the extent of the area influenced by an individual storm; those of May 1940, August 1947 and December 1957 were particularly widespread.

Water balance

A simple water balance study has been undertaken to assess the reliability of the runoff records, and also to give an indication of the long-term yield at ungauged sites. The water balance compares the catchment losses, as determined by the rainfall minus runoff, with the losses indicated by evapotranspiration and interception. As the rainfall in the upper Mahaweli is greater than the potential transpiration, and there are negligible groundwater losses, the water balance approach is particularly appropriate.

As mentioned above, the long-term mean annual rainfalls calculated from the isohyetal map were derived from records covering the standard thirty year period 1931 to 1960. It is, however, unnecessary to adjust the long-term mean when comparing catchment rainfall with a runoff record of 10 years or more for the following reasons. Examination of a long term rainfall record has suggested that the annual falls are normally distributed, and that the persistence from year to year is small. P. S. Harihara Ayyar (7) has also indicated that there is little evidence of serial dependence in both the annual and seasonal rainfall. We have therefore assumed that the rainfall from year to year is independent. The data also indicate that the coefficient of variation of catchment rainfall is approximately 15 per cent; thus, for a record of more than 10 years the standard error of estimate of the mean annual rainfall will be less than 5 per cent. For a runoff record of more than 10 years, the mean catchment rainfall should therefore be based on the isohyetal

MAXIMUM RECORDED DAILY RAINFALL

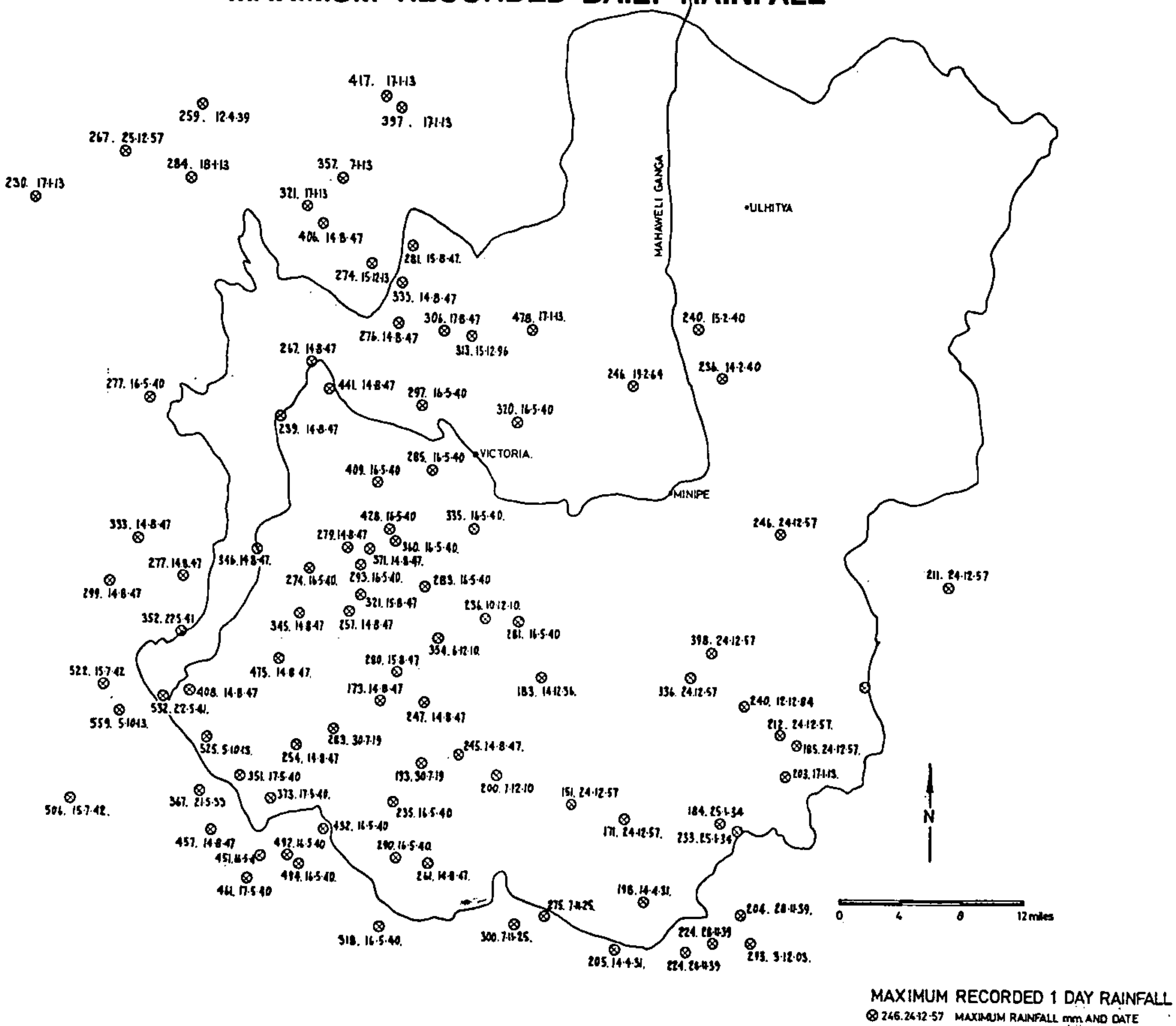


Figure 6

map. In contrast for records shorter than 10 years the actual rainfall, previously estimated by the Irrigation Department using Thiessen polygons, should be used when comparing rainfall with runoff.

The relationship between evaporation and altitude is illustrated in Figure 7 where the Penman estimates of E_o, discussed earlier, are plotted against the elevation of the individual stations. Note that both average values from the long-term climatological stations and single year values, from the recently established agrometeorological stations, are plotted. The points show that there is a relationship between evaporation and altitude and suggest that for catchments at the median elevation of 1200 m, the annual open water evaporation is about 1800 mm/year.

The estimates of potential transpiration, E_t, given in Table 2 were calculated using an assumed albedo of 0.25. Although this value is usually assumed for grassland, it can be applied in the north-east of the study area where the vegetation is characterised by parkland and scrub. Alternative estimates have also been made using an albedo of 0.15 to take account of the lower reflection typical of the dense forest of the upper catchment and to make some allowance for the evaporation of rainfall intercepted by the tree canopy.

The relationship between E_t and altitude is drawn in Figure 7 for each albedo, but for clarity the data points have been omitted. On this evidence, we have assumed that the potential transpiration for catchments at the median elevation of about 1200 m should be between 1400 and 1600 mm/year. As the rainfall is sufficiently high in the upper catchment and there is no prolonged dry season, we consider that the actual catchment losses should be of this order.

The mean annual runoff for each of the gauging stations within the study area was calculated from the data available up to 1976 when Polgolla was commissioned. These results are summarised with the estimates of catchment rainfall in Table 3, and illustrated in Figure 8 where the mean annual rainfall is plotted against mean annual runoff.

EVAPORATION AGAINST ALTITUDE

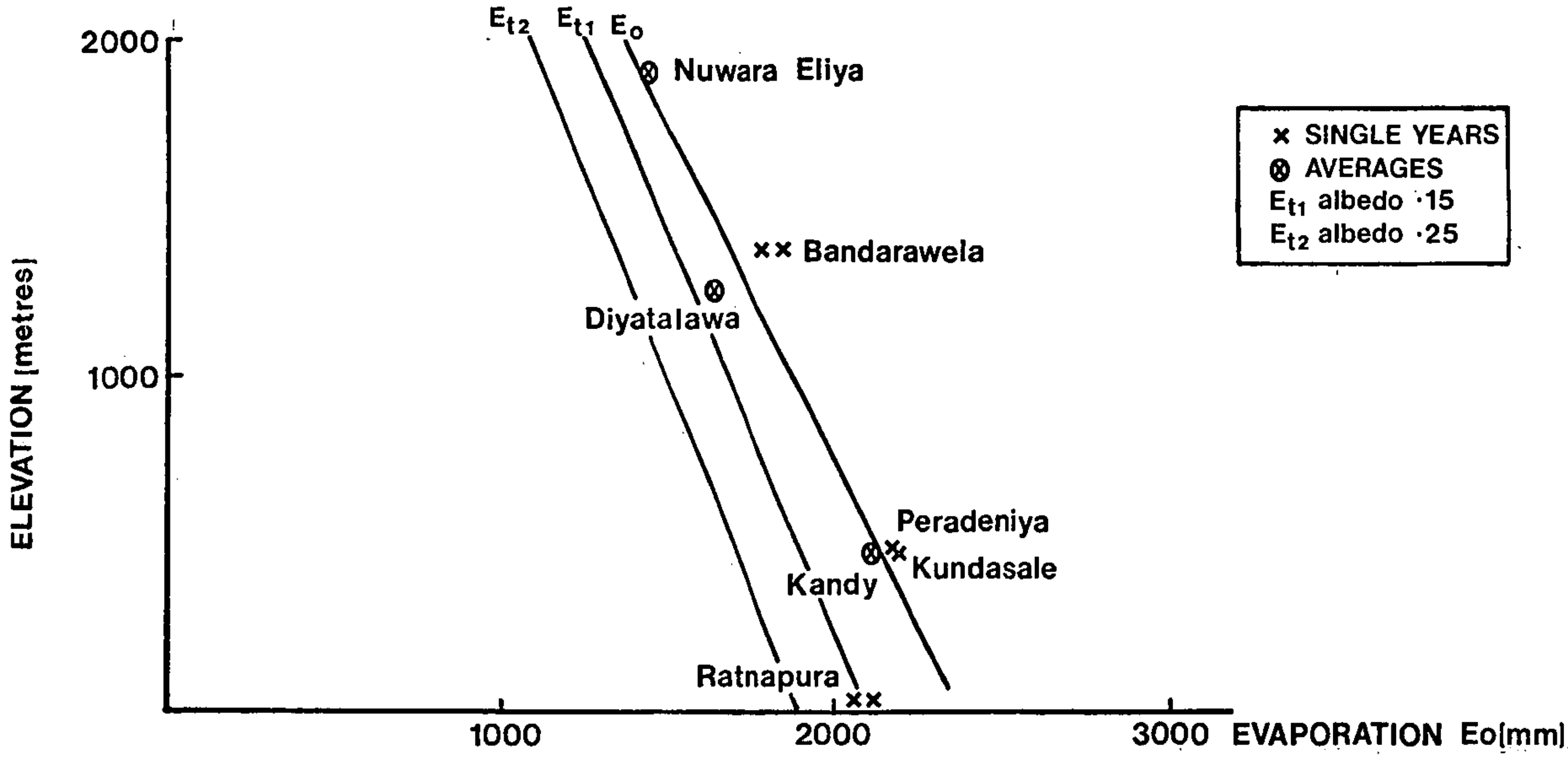


Figure 7

TABLE 3

COMPARISON OF MEAN ANNUAL RAINFALL AND RUNOFF

River	Station	Flow records Period from	to	No. of years	Mean runoff (mm)	Corresponding rainfall (mm)	Long-term rainfall[1] (mm)
Mahaweli Ganga	Peradeniya	1944/5	1974/5	31	1794		3118
"	Gurudeniya	1944/5	1974/5	31	1742		2934
"	Randenigala	1955/6	1974/5	20	1687		2713
"	Weragamtota	1945/6	1974/5	30	1389		2439
	Manampitiya	1944/5	1974/5	31	975		2410
Hulu Ganga	Teldeniya	1954/5	1974/5	21	1140		3273
Galmal Oya	Moragamulla	1963/4	1974/5	12	1182		2731
Maha Oya	Hanguranketa	1954/5	1959/60	6	967	2546[3]	2553
Uma Oya	Talawakanda	1957/8	1974/5	18	646		2064
Loggal Oya		1955/6	1964/5	10	1307	2403[3]	2295
Ulhitiya Oya		1953/4	1959/60		398	2404[3]	2150
Maduru Oya	Kandegama	1951/2	1956/7		604	2068[3]	2214[2]
Gallodai Aru		1945/6	1974/5	30	671		2144[2]

Note: [1] derived from Figure 5, except where stated otherwise.
[2] taken from (3)
[3] rainfall for the period corresponding to the runoff record, from (3)

River	Catchment between		Mean runoff (mm)	Long-term rainfall (mm)
Mahaweli Ganga	Peradeniya	Gurudeniya	1474	2099
Mahaweli Ganga	Gurudeniya	Randenigala	1495	2404
Mahaweli Ganga	Randenigala	Weragamtota	1090	2068

MEAN ANNUAL RAINFALL AND RUNOFF

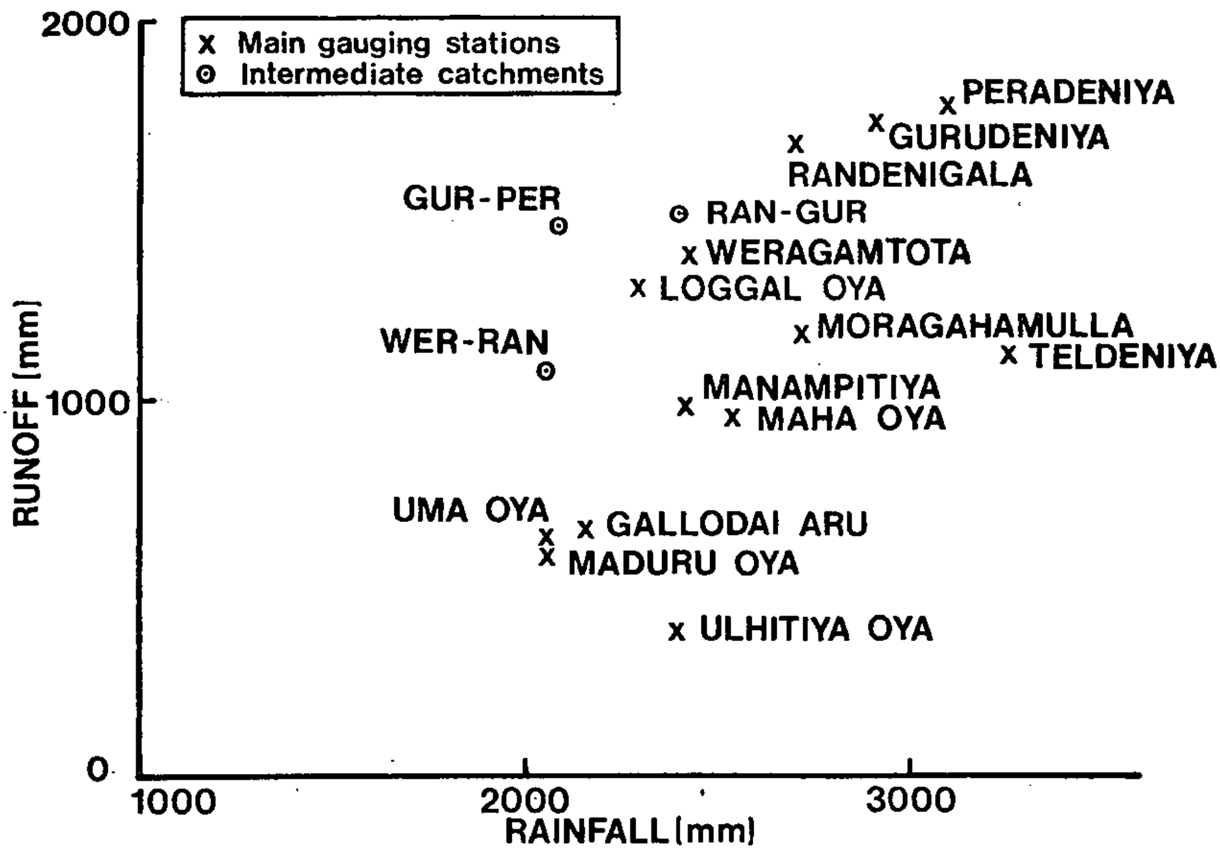


Figure 8

The water balance analysis indicates that the runoff records are reasonable, with the exception of the Ulhitiya Oya and Hulu Ganga at Teldeniya where the runoff appears to be underestimated. At the other stations, there is some scatter in the points but this can be explained by errors in the estimation of rainfall, particularly in those areas where the raingauge network is sparse. The apparently low runoff at Manampitiya can be explained by losses to groundwater and to irrigation, downstream of Weragamtota. We consider that with the exception of the two stations mentioned above and Victoria, there appears to be no reason to reject the data from any other station on water balance grounds.

We have also examined the relationship between the rainfall and runoff of some of the intermediate catchments on the Mahaweli Ganga; here the runoff is estimated by the difference between the records over a contemporary period at an upstream and downstream gauge. The rainfall is again estimated from the isohyetal map. This is a severe test of the runoff data as any systematic errors in measurement will be compounded by taking differences. The resulting points, also shown in Figure 8, increase the scatter but could still be considered consistent with the other observations, with the possible exception of the small area between Peradeniya and Gurudeniya.

The water balance approach also allowed estimates of the catchment losses and hence the long-term yield at ungauged, or poorly gauged, sites of importance to be made. Such estimates were necessary for the tributaries crossing the right bank canal and also the Ulhitiya Oya.

As no evaporation data were available for this part of the study area, the catchment losses had to be estimated from the observed rainfall and runoff data at Talawakanda and on the Gallodai Aru (see Table 3). The observed losses range from 1418 mm at Talawakanda, to 1473 mm on the Gallodai Aru. A reasonable, though conservative, estimate of the losses in this region was made as 1500 mm. The long-term yield thus deduced for the catchments of interest is given in Table 4.

TABLE 4

ESTIMATES OF LONG TERM YIELD

Site	Catchment area (km)	Mean annual rainfall (mm)	Deduced runoff (mm)
Ulhitiya	290	2150	650
Diayabawa, right-bank canal	18	2150	650
Hepola Oya, " " "	140	2160	660
Loggal Oya, "	250	2235	735

It must be appreciated that these values are deduced from the differences between two large numbers. Of these the estimates of the catchment losses could be subject to considerable error, and although there are few raingauges in these catchments, we can assume that the error in mean annual rainfall should be smaller, so that the runoff quoted in Table 4 must be regarded as tentative and should be supported by short-term gauging.

Estimation of flow records

For the reservoir operation model, a long sequence of monthly natural river flows are required at the following sites:

(1) Polgolla,
(2) Victoria dam,
(3) Randenigala dam,
(4) Minipe anicut,
(5) the major tributaries (except Badulu Oya) crossing the right-bank canal,
and (6) Ulhitiya dam.

From these sites, reliable recorded data are available only at Randenigala and these data cover too short a period for use in the operation model. The record at Gurudeniya however, covers a period of 31 years, up to the commissioning of Polgolla in 1976. In view of the time available for this study, we have accepted that this record is sufficiently long for the operational studies, and consequently we have based the extension of the short term records on these data.

The 'tributary inflow' between Gurudeniya and Randenigala can be calculated by the difference in flow between the upstream and downstream gauging stations. The proportion of this tributary inflow that joins the main river above the Victoria dam site can then be estimated from the ratio of the relative catchment areas and the long-term catchment rainfall, minus losses. Similarly, as the major tributary between Randenigala and Minipe is the Uma Oya, the tributary inflows between these two points can be estimated from the flows at Talawakanda multiplied by a factor, which again depends on the catchment area, rainfall and losses. It was therefore decided that the flow records at Randenigala and Talawakanda should be extended to the period covered by the Gurudeniya record. To avoid the possibility of generating negative inflows between Gurudeniya and Randenigala, the recorded 'tributary inflows' were used instead of the Randenigala flows. The latter can then be found by addition, and used with the generated Uma Oya flows to extend the record at Minipe.

As the catchment area between Polgolla and Gurudeniya is small, the Polgolla flows have been deduced from the Gurudeniya records multiplied by a factor of 0.926, based on the relative catchment areas and the long-term rainfalls minus losses.

The model chosen for extending the flow data at sites with incomplete records is a multivariate lag-one Markov model. This type of model allows cross-correlations between concurrent flows at a number of gauging stations, as well as lag-one serial correlations within the individual station records, to be preserved in the extended flow sequences. A more detailed description of the model is given in the Annex.

The model was tested by generating 50 sequences of flow data over the period for which flow records were available at Talawakanda and Randenigala. The historical data at each site were then tested statistically to see whether they could have reasonably come from the population of the 50 generated sequences. The more important results of the comparison are shown in Table 5.

A statistical significance test indicated that the null hypothesis that the historical data could have come from the population of the generated sequences could not be rejected at the 5 per cent confidence level in any of the comparisons made. The model was thus accepted and used to extend the short-term records. An example of the generated and recorded data is given in the computer output, shown in Table 6, (Year 1 is 1944/45).

TABLE 5

COMPARISON OF GENERATED AND HISTORIC DATA

	Month	Generated flows (m^3/s) mean	Generated flows (m^3/s) standard deviation	Historic flows (m^3/s) mean
Tributaries				
	O	47.64	4.99	44.98
	N	68.88	6.00	66.43
	D	96.29	10.49	92.12
	J	75.15	5.95	72.45
	F	66.19	10.56	60.21
	M	26.52	3.57	24.18
	A	37.53	4.02	34.81
	M	33.54	4.04	30.99
	J	24.09	2.29	27.34
	J	33.25	3.38	32.71
	A	35.68	4.39	32.99
	S	30.05	3.59	28.72
Annual		47.90	2.35	45.66
Uma Oya				
	O	9.54	1.01	9.52
	N	15.02	1.69	14.82
	D	19.46	2.46	19.73
	J	16.62	2.29	16.14
	F	11.82	1.65	12.00
	M	8.98	1.32	9.01
	A	11.21	0.88	11.29
	M	10.33	1.29	10.47
	J	5.95	0.46	5.91
	J	4.94	0.58	4.80
	A	4.78	0.45	4.71
	S	5.69	0.57	5.72
Annual		10.36	0.71	10.34

Notes: Tributaries are the tributaries between Gurudeniya and Randenigala

The statistics for the generated flows are taken from a sample of 50 sequences of 18 years.

TABLE 6

RECORDED, SYNTHETIC AND DEDUCED MONTHLY FLOWS : MAIN RIVER

MEAN MONTHLY DISCHARGE (M3/S)

VICTORIA (DEDUCED)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AV
1	154.77	162.79	192.84	65.15	39.01	18.60	42.76	25.04	70.62	33.84	113.98	57.77	81.43
2	156.84	141.36	173.28	76.70	67.57	42.51	92.91	27.69	60.04	40.36	91.99	67.21	85.70
3	77.33	139.12	214.99	139.87	37.93	65.47	55.47	26.96	50.27	59.89	334.86	68.39	105.88
4	67.50	31.15	42.11	36.76	24.70	6.27	19.92	33.41	153.08	146.94	43.58	21.22	52.22
5	72.97	72.71	81.95	109.31	34.06	38.51	50.31	62.20	132.17	147.51	157.91	139.86	91.54
6	80.34	80.71	88.27	54.56	53.50	54.91	35.33	87.14	69.31	121.61	109.30	221.62	88.05
7	109.79	85.49	58.06	156.52	77.44	37.03	62.92	37.69	389.01	130.33	13.98	103.99	105.19
8	152.38	222.14	76.22	115.66	50.81	27.89	65.15	326.42	212.19	113.86	118.89	60.98	130.21
9	294.99	98.05	90.38	38.01	41.51	34.02	64.88	29.37	43.55	148.38	83.07	85.18	87.62
10	119.87	89.77	100.99	97.43	64.16	79.47	105.32	124.77	113.75	97.43	175.42	115.88	107.19
11	200.58	66.60	194.21	130.86	87.13	54.21	102.08	192.82	301.47	134.35	84.62	103.59	139.38
12	105.83	107.97	66.51	42.61	26.54	32.12	30.36	49.53	285.99	94.99	102.20	100.45	67.17
13	165.80	217.41	110.69	76.33	73.21	34.95	35.45	50.82	209.60	218.08	85.46	68.44	112.29
14	74.26	216.50	561.14	153.96	55.72	80.20	91.73	121.93	115.87	229.91	133.72	56.58	157.63
15	201.17	184.66	91.92	53.73	41.58	31.66	65.92	64.48	258.83	218.32	87.86	94.58	116.23
16	157.62	145.00	98.53	104.60	195.18	55.27	78.50	80.65	111.17	135.96	119.87	209.32	124.39
17	165.80	219.24	80.72	54.96	47.29	28.82	35.84	115.28	88.79	100.79	160.01	93.89	99.37
18	97.28	130.12	123.92	88.82	59.24	25.84	49.72	135.44	78.22	121.52	87.04	158.45	96.30
19	169.66	124.35	94.72	110.14	77.29	38.61	64.17	53.45	80.15	112.27	100.88	95.78	93.54
20	156.54	146.85	176.42	133.30	103.59	63.31	41.01	58.12	56.40	119.26	115.51	134.65	108.75
21	92.01	179.30	65.45	46.91	82.87	28.43	91.98	205.11	126.47	51.82	118.00	75.48	98.74
22	140.40	137.17	137.40	81.05	43.91	40.38	75.74	47.63	48.22	69.68	73.73	154.34	87.47
23	162.26	152.62	89.77	74.96	74.39	51.45	37.08	31.20	73.83	93.32	84.71	50.01	81.30
24	194.30	189.21	182.51	66.29	31.47	43.18	46.49	71.98	109.11	231.66	151.17	148.68	122.17
25	147.60	153.28	120.20	92.21	65.76	23.16	90.22	127.29	158.77	81.75	62.60	112.86	103.06
26	153.99	102.51	127.05	118.02	130.45	47.54	88.56	77.04	115.57	106.81	171.62	90.92	110.84
27	157.79	165.27	195.92	144.27	53.63	47.22	92.06	105.12	141.60	159.74	179.57	261.34	141.96
28	178.44	98.62	208.19	81.86	45.96	14.65	54.91	174.95	61.01	153.25	116.49	75.71	105.42
29	221.00	230.66	220.74	67.29	39.25	20.29	36.43	20.42	55.14	87.01	163.98	68.13	102.53
30	64.99	129.31	146.16	70.70	37.12	42.92	92.66	122.05	165.09	216.94	204.62	175.95	122.38
31	145.81	71.82	88.49	83.87	41.62	43.57	63.57	77.93	227.55	98.16	196.03	171.16	109.13
MEAN	143.22	139.09	139.35	89.25	61.45	40.40	63.95	89.19	134.32	125.06	123.63	111.11	105.00
S D	51.78	51.59	93.54	34.76	34.15	17.34	24.52	66.88	86.10	54.19	59.47	55.14	20.92

TABLE 6

(continued.....)

MEAN MONTHLY DISCHARGE (M3/S)

GURUDENIYA (RECORDED)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	..
1	124.01	132.93	97.73	23.48	14.73	14.95	24.84	14.75	61.11	25.63	80.09	32.48	53.90
2	117.19	107.90	98.98	28.66	18.95	26.25	55.85	18.12	47.27	32.17	65.28	54.01	55.89
3	59.25	93.48	155.62	90.09	18.95	57.04	39.14	22.00	38.49	46.10	284.59	58.03	80.23
4	53.30	18.46	11.89	13.71	6.94	3.43	8.24	23.39	137.92	131.32	32.34	13.68	37.89
5	57.40	49.87	57.06	49.62	14.84	18.27	41.18	54.01	123.11	128.97	131.06	99.91	68.78
6	67.18	59.90	66.96	25.74	24.50	29.11	19.40	60.07	54.88	95.78	90.43	204.50	66.54
7	68.27	53.95	24.95	79.49	43.58	27.75	47.27	25.73	373.74	116.20	9.32	93.88	82.09
8	127.86	177.65	56.33	77.82	34.52	20.56	59.87	301.27	188.47	91.53	100.90	43.87	106.72
9	220.02	67.68	34.78	24.21	18.66	18.46	34.41	16.94	33.25	126.85	71.96	69.87	61.42
10	85.19	61.40	44.56	40.04	27.53	35.23	67.77	84.90	79.24	80.40	145.68	85.67	69.80
11	178.61	63.64	127.98	73.15	53.50	44.29	64.77	161.85	282.55	125.68	71.28	83.57	110.99
12	89.04	91.64	49.70	28.07	24.24	28.52	25.86	44.52	252.76	83.15	94.02	91.56	75.26
13	145.08	172.47	71.54	44.60	40.98	25.26	31.18	45.45	178.81	196.51	78.62	58.37	90.74
14	68.22	156.47	400.67	92.29	33.62	64.91	51.77	89.55	100.00	205.72	108.61	48.80	118.38
15	146.47	131.55	53.21	35.51	31.27	26.42	48.57	53.69	247.25	201.92	79.64	82.98	94.87
16	130.07	100.45	60.29	47.18	74.03	33.13	60.26	67.51	103.76	119.71	103.08	192.04	90.96
17	145.22	178.56	59.76	32.94	27.95	21.55	29.54	109.06	75.59	68.05	149.73	79.55	63.12
18	80.88	97.73	75.90	42.23	23.17	18.10	34.72	109.51	66.33	99.46	73.63	123.45	70.42
19	149.47	96.85	55.85	53.69	30.47	24.81	46.42	45.51	71.62	99.71	64.45	84.73	70.30
20	132.40	99.54	115.18	55.93	35.03	27.73	22.54	37.27	45.60	103.00	99.66	121.63	74.63
21	69.10	153.18	48.54	20.73	21.16	13.91	56.05	172.21	114.04	44.55	100.22	62.45	73.01
22	112.94	96.06	78.56	34.83	17.47	21.64	44.29	23.59	27.30	41.97	47.24	143.24	57.43
23	122.43	101.98	47.63	35.37	32.37	28.57	22.74	21.64	59.81	75.19	67.20	37.47	54.37
24	167.97	126.63	136.53	33.22	17.13	25.46	31.35	56.58	101.16	200.87	135.88	128.03	96.90
25	119.91	121.52	71.46	39.87	24.13	16.71	58.65	112.20	149.98	73.29	50.55	95.41	77.81
26	113.28	73.66	84.85	61.06	55.82	25.15	57.60	56.58	95.69	79.61	147.58	70.12	76.75
27	120.90	129.11	133.19	69.04	32.60	26.88	71.20	74.74	121.89	134.66	139.70	222.68	106.38
28	148.65	78.79	105.44	29.71	17.28	10.87	32.94	143.24	43.47	128.15	92.21	69.44	75.01
29	187.34	162.56	113.93	38.80	20.08	13.14	21.75	13.79	40.87	56.27	125.80	43.95	69.86
30	43.87	80.54	86.60	41.66	25.09	29.85	59.87	90.31	142.08	209.60	169.72	154.66	94.49
31	119.54	53.38	52.36	39.68	20.67	20.87	39.48	52.59	202.32	68.31	171.68	143.58	82.04
MEAN	115.84	102.95	86.39	45.24	28.43	25.77	42.24	71.08	118.08	106.82	103.29	93.34	78.29
S D	42.61	41.62	67.72	20.59	13.85	12.28	16.40	60.66	83.41	52.46	51.46	50.91	18.36

MEAN MONTHLY DISCHARGE (M3/S)

RANDENIGALA (RECORDED AND GENERATED)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AV
1	175.27	182.69	256.25	92.93	55.20	21.04	54.70	31.90	76.96	39.31	136.58	74.63	99.79
2	183.27	163.66	222.82	108.72	99.98	53.35	117.61	34.06	68.55	45.82	93.13	76.01	105.58
3	89.38	169.54	254.57	173.06	50.59	71.09	66.36	30.26	58.12	69.08	368.37	75.30	122.98
4	76.96	39.61	62.26	52.13	36.53	8.17	27.70	40.08	163.19	157.35	51.07	26.25	61.78
5	83.34	87.94	98.55	149.11	46.87	52.01	56.40	67.67	138.21	159.86	175.81	164.83	106.72
6	69.12	94.59	102.47	73.77	72.84	72.11	45.96	105.18	78.92	138.84	121.88	233.03	102.39
7	124.13	106.52	80.13	207.87	100.02	43.22	73.35	44.99	399.19	139.75	17.09	110.73	120.58
8	168.72	251.79	89.46	140.89	61.66	32.78	102.01	343.19	228.00	128.74	130.87	72.38	145.88
9	344.96	118.30	127.45	47.22	56.74	44.39	85.20	37.66	50.41	162.74	90.47	95.40	105.08
10	142.99	108.69	138.60	135.68	88.58	108.97	132.02	151.35	136.76	108.78	195.25	137.68	132.11
11	215.22	101.91	238.37	169.33	109.55	60.82	126.96	213.47	314.08	139.47	93.52	116.93	158.30
12	117.02	118.86	77.71	52.31	28.07	34.52	33.36	52.87	309.82	102.89	107.64	105.37	95.12
13	179.61	247.38	137.13	97.48	94.70	41.40	39.96	54.40	230.13	232.45	90.03	75.16	126.65
14	78.28	256.52	668.13	195.07	70.46	90.40	118.38	143.53	126.45	246.04	150.46	61.77	183.79
15	237.63	220.07	117.73	65.87	48.46	35.15	77.48	71.68	266.55	229.25	93.34	102.32	130.46
16	175.98	174.71	124.01	142.87	275.95	70.04	90.65	89.41	116.11	148.45	131.06	220.84	145.57
17	179.52	246.36	94.70	69.64	60.18	33.67	41.72	119.43	97.59	109.29	165.86	103.45	110.20
18	108.21	151.71	155.93	119.88	83.29	31.01	59.73	152.73	86.15	136.22	95.98	181.79	113.55
19	183.12	142.68	120.64	147.77	108.49	47.80	76.01	58.74	85.84	120.64	111.84	104.81	109.03
20	172.64	178.39	217.24	184.87	149.30	87.03	53.33	72.02	63.61	130.10	126.08	143.33	131.49
21	107.28	196.71	110.05	64.37	124.01	38.12	115.94	228.71	134.75	56.67	129.85	84.17	115.89
22	158.71	164.57	176.63	111.86	61.54	52.87	96.71	63.66	62.16	98.16	91.39	161.74	107.50
23	168.81	186.37	117.87	101.36	102.41	66.69	45.64	37.58	83.18	105.41	96.37	58.37	99.25
24	211.86	229.59	213.16	88.33	41.04	55.00	56.58	82.24	114.41	252.19	161.37	162.44	139.02
25	166.07	174.45	152.67	127.10	95.10	27.50	111.27	137.35	164.62	67.40	70.63	124.49	119.90
26	181.13	121.75	155.19	155.99	180.20	62.47	109.20	90.68	128.83	124.95	187.65	104.78	133.57
27	182.38	189.38	237.75	194.42	67.66	60.77	105.97	125.37	154.74	176.46	206.14	287.11	165.68
28	198.30	111.84	276.69	116.62	65.08	17.16	69.55	195.09	72.70	169.98	132.68	81.56	125.69
29	243.44	276.06	291.95	86.29	52.02	25.06	45.22	24.84	64.65	107.50	189.43	84.25	124.31
30	79.07	161.82	165.86	90.06	45.14	51.63	114.53	143.21	180.43	221.83	227.89	190.14	140.97
31	163.32	84.11	112.57	113.34	55.59	58.71	79.64	94.82	244.37	118.07	212.26	189.55	127.19
MEAN	161.47	163.18	174.66	118.59	83.46	50.16	78.42	101.26	145.14	137.22	137.19	122.96	122.81
S D	58.77	59.14	111.99	45.65	49.23	22.42	30.68	71.60	88.19	55.86	65.26	58.54	23.44

TABLE 6

(continued....)

TABLE 6

(continued....)

MEAN MONTHLY DISCHARGE (M3/S)

MINIPE (DEDUCED)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	
1	185.05	195.51	265.02	98.79	58.93	23.43	61.50	36.80	80.72	40.99	140.45	79.22	105.53
2	196.05	185.10	252.82	121.51	113.39	68.29	129.75	40.97	73.36	47.31	96.99	80.90	117.20
3	94.35	176.36	275.29	208.63	66.98	98.51	83.25	42.78	62.86	74.00	375.66	79.49	136.58
4	63.18	45.35	69.67	56.22	42.91	10.69	35.52	45.64	172.59	163.86	53.79	29.21	67.47
5	91.64	103.50	123.26	179.10	52.63	57.08	66.46	77.22	141.80	152.93	179.16	167.18	116.83
6	103.24	105.84	131.03	84.14	87.55	98.81	64.06	118.78	64.94	143.25	126.75	241.69	115.84
7	134.53	119.55	92.99	221.29	114.91	51.44	81.38	47.94	403.65	142.30	18.07	115.24	128.61
8	182.17	276.15	100.40	157.37	76.98	39.51	110.03	354.97	235.33	131.73	134.24	76.49	156.28
9	359.08	130.27	142.23	56.17	60.71	48.48	94.34	42.39	55.04	171.91	97.45	99.18	113.10
10	147.27	114.37	147.79	144.91	95.94	116.23	142.28	156.50	139.98	110.59	198.46	140.76	137.92
11	244.99	129.04	265.91	227.54	148.11	104.46	156.58	253.51	331.12	145.22	99.81	125.81	167.68
12	127.37	130.80	83.82	56.87	36.55	39.98	40.00	59.32	317.12	106.85	111.43	110.76	101.74
13	189.69	273.94	154.22	119.27	111.55	49.26	52.63	72.40	240.86	240.55	94.26	81.82	140.05
14	91.14	288.94	731.87	232.99	90.06	106.14	135.69	159.65	134.10	252.54	157.18	65.37	203.89
15	247.43	238.19	132.22	77.81	56.49	39.24	90.00	83.07	276.44	236.48	97.50	107.41	140.19
16	189.81	193.30	136.99	164.38	316.13	87.57	112.45	102.24	122.86	159.37	136.35	228.20	162.80
17	192.39	268.82	103.31	88.65	78.64	46.89	58.64	140.93	106.78	116.23	172.91	110.72	123.74
18	115.35	168.80	177.60	139.53	97.53	40.71	71.12	165.97	92.77	142.27	101.54	187.96	125.18
19	194.48	152.72	134.82	169.76	125.42	58.56	89.48	70.74	93.17	125.34	115.09	110.12	120.14
20	186.27	200.53	244.89	212.84	165.40	104.72	63.02	80.56	70.33	136.05	131.14	148.74	145.38
21	111.92	205.90	118.12	68.24	132.91	42.69	128.74	247.91	143.61	51.47	135.89	88.52	123.91
22	171.09	177.11	201.06	126.30	69.28	62.44	108.71	71.41	67.31	91.81	95.58	170.18	117.69
23	200.52	199.17	129.74	106.83	106.34	71.14	52.72	40.88	85.61	105.50	99.73	61.79	105.08
24	225.17	254.94	234.54	100.59	47.24	64.53	63.78	87.55	118.19	255.97	165.56	167.40	148.79
25	175.35	191.06	163.87	141.47	103.57	32.07	120.61	145.54	170.19	90.53	75.69	130.09	128.34
26	196.18	132.08	173.43	179.22	204.68	75.56	123.95	113.18	135.71	132.02	191.55	111.25	147.40
27	168.81	204.00	253.56	208.15	75.59	66.98	112.98	132.19	158.96	180.88	210.05	291.40	173.63
28	202.46	117.28	292.91	128.08	73.34	19.85	83.70	207.93	77.82	171.99	134.15	84.63	132.84
29	261.10	296.99	316.99	102.29	65.98	34.63	56.87	30.69	69.52	111.92	193.85	68.44	135.94
30	89.05	184.54	218.03	104.01	53.81	63.31	131.55	158.51	188.49	227.88	238.32	205.40	155.33
31	170.84	94.22	162.14	152.86	65.74	77.27	95.25	105.06	255.48	126.03	223.01	203.66	144.30
MEAN	172.52	179.24	195.31	136.64	96.69	61.31	90.87	112.75	151.83	142.19	142.05	128.74	134.18
S D	60.92	63.91	121.05	53.59	55.86	27.70	32.79	75.37	89.71	56.87	66.29	59.97	26.53

TABLE 6

(continued....)

MEAN MONTHLY DISCHARGE (M3/S)

POLGOLLA (DEDUCED)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	
1	114.89	123.15	90.54	21.75	13.64	13.85	23.01	13.67	56.62	23.74	74.19	30.09	49.93
2	108.56	99.96	91.69	26.55	17.55	24.32	51.74	15.79	43.79	29.80	60.47	50.03	51.77
3	54.88	86.60	144.16	83.46	17.55	52.64	36.26	20.39	35.65	42.71	263.64	53.76	74.33
4	49.38	17.11	11.02	12.70	6.43	3.17	7.63	21.67	127.77	121.65	29.96	12.67	35.10
5	53.16	46.20	52.86	45.96	13.75	16.92	38.15	50.03	114.05	119.48	121.42	92.56	63.71
6	62.23	55.49	62.05	23.85	22.69	26.97	17.97	55.65	50.84	98.73	83.77	189.45	61.64
7	81.78	49.98	23.11	73.64	40.38	25.71	43.79	24.77	346.23	107.64	9.63	85.97	76.05
8	118.45	164.58	52.16	72.10	31.98	19.05	55.46	279.09	174.60	84.79	93.48	40.64	98.87
9	203.82	62.70	32.22	22.43	17.29	17.11	31.88	15.69	30.80	117.51	66.66	64.72	56.90
10	76.92	56.88	41.29	37.10	25.50	32.64	62.78	78.65	73.41	74.48	134.96	79.36	64.66
11	165.47	58.95	118.56	67.77	49.56	41.03	60.00	149.94	261.75	117.35	66.04	77.42	102.82
12	82.48	84.90	46.04	26.00	22.46	26.42	23.95	41.24	234.15	77.03	87.10	84.82	69.72
13	134.41	159.78	66.27	41.32	37.96	23.40	28.89	42.11	165.65	182.05	72.83	54.07	84.06
14	63.20	144.95	371.18	85.50	31.14	60.13	47.96	82.96	92.64	190.58	100.61	45.20	109.67
15	135.69	121.86	49.30	32.90	28.96	24.48	44.99	49.74	229.06	187.06	73.77	75.87	87.89
16	120.50	93.06	55.86	43.71	68.58	30.70	55.83	62.55	96.13	110.90	95.50	177.93	64.27
17	134.54	165.42	55.36	30.51	25.89	19.97	27.36	101.03	70.02	81.57	138.71	73.70	77.01
18	74.93	90.54	70.31	39.12	21.46	16.76	32.16	101.45	61.44	92.14	68.21	114.36	65.24
19	138.47	89.73	51.74	49.74	28.23	22.98	43.00	42.16	66.35	92.38	78.23	78.50	65.13
20	122.65	92.22	106.70	51.82	32.45	25.68	20.88	34.53	42.24	95.42	92.32	112.58	69.13
21	64.01	141.91	44.97	19.20	19.60	12.86	51.92	159.54	105.65	41.27	92.85	57.85	67.54
22	104.63	88.99	72.78	32.27	16.19	20.04	41.03	21.85	25.29	38.88	43.76	132.70	53.20
23	113.42	94.47	44.13	32.77	29.99	26.47	21.07	20.04	55.41	69.56	62.26	34.71	50.37
24	155.60	119.16	126.48	30.77	15.87	23.59	29.04	52.42	93.71	186.09	125.88	118.61	89.77
25	111.08	112.58	66.22	36.94	22.35	15.48	54.33	103.95	138.94	57.90	45.83	88.39	72.08
26	104.94	68.24	78.60	56.56	51.71	23.30	53.36	52.42	88.65	73.75	135.71	64.96	71.10
27	112.00	119.61	123.39	63.96	30.20	24.90	65.96	69.24	112.92	124.75	129.42	206.29	98.55
28	137.71	72.99	97.68	27.52	16.00	10.07	30.51	132.70	40.27	118.72	85.42	64.33	69.49
29	173.55	150.59	105.55	35.94	18.60	12.17	20.15	12.78	37.86	52.13	116.54	40.72	64.71
30	40.64	74.61	80.23	38.59	23.24	27.65	55.46	83.67	131.62	194.17	157.23	143.27	87.53
31	110.74	49.45	48.51	36.76	19.15	19.34	36.57	48.72	187.43	63.28	159.04	133.01	76.00
MEAN	107.31	95.38	80.03	41.91	26.33	23.87	39.13	65.85	109.39	98.95	95.69	85.47	72.53
S D	39.47	38.56	62.74	19.07	12.83	11.38	15.19	56.19	77.27	48.60	47.67	47.17	17.01

Because insufficient data were available, a different approach had to be adopted for the right bank tributary gauging stations and the Ulhitiya Oya. Given that the long-term yield of the relevant catchments can be estimated, the problem becomes one of estimating the distribution of flows within each year and also from year to year.

As the percentage monthly distribution of the flows recorded at the gauging stations in the north-east of the study area appeared to be consistent, these distributions were used to estimate the mean monthly flow from the long-term yield at each of the pertinent sites. We then estimated the variability about the monthly means by assuming a coefficient of variation of 70 per cent; this value was based on the limited available data. By analogy with values of the serial and cross correlation coefficients calculated earlier at Gurudeniya and Talawakanda, it was possible to use a version of the Markov model to generate monthly flows for the Ulhitiya Oya and the tributaries entering the right bank canal. While the data thus generated should be used with caution, since the assumptions made in their derivation are tenuous, we believe that they indicate the flows that might have occurred for the period common to the data recorded at Gurudeniya. The data are shown in Table 7.

The flow generation model includes a random element with zero mean and with variance estimated during the fitting of the model to the observed records. This ensures that the variance of the flows about the expected mean is preserved in the generated data. Each time a flow sequence of a given number of years is simulated, the actual values obtained will be different because of variations in the sequence of random elements. Each simulated sequence can therefore be regarded as one realisation from a population of sequences each of which has basic statistical properties similar to those of the observed flow sequence. Clearly, the more simulations that are performed, the more likely it is that any statistic derived from the simulated sequence will approach the population value.

The generated flow data given in Tables 6 and 7 are internally consistent but they are derived from just one realisation of the model.

TABLE 7

RECORDED, SYNTHETIC AND DEDUCED MONTHLY FLOWS: ULHITIYA AND RIGHT-BANK CANAL

ESTIMATED MEAN MONTHLY DISCHARGE(M3/S)

ULHITIYA OYA

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AV
1	.45	14.39	10.26	5.65	15.88	14.24	3.75	2.40	6.75	.21	.39	.03	6.20
2	.48	5.37	7.95	6.57	5.62	4.60	7.86	3.93	1.36	.16	.23	.08	3.69
3	.29	2.21	11.89	38.87	8.83	23.65	10.37	3.18	1.75	.10	.33	.11	8.46
4	.22	1.53	10.26	5.67	5.12	2.46	1.79	4.45	1.34	.23	.08	.04	2.77
5	1.73	2.99	9.35	30.19	4.51	5.39	3.00	6.87	5.60	.20	.12	.14	5.84
6	.74	4.49	13.57	14.73	36.68	8.50	6.77	6.11	2.32	.26	.16	.04	7.87
7	.60	5.28	15.79	14.63	18.36	2.40	4.11	3.61	3.54	.21	.04	.12	5.72
8	.70	6.24	9.84	48.74	12.47	5.01	5.37	2.55	3.57	.17	.41	.08	7.93
9	1.76	5.07	6.27	10.70	8.20	2.54	7.82	2.37	4.85	.36	.13	.05	4.18
10	.69	9.11	12.27	13.46	9.37	4.03	6.70	2.99	1.80	.13	.39	.04	5.10
11	.28	1.82	25.83	14.81	16.46	4.30	9.26	7.74	4.81	.30	.17	.12	7.16
12	.23	2.84	4.07	11.23	19.22	14.23	7.81	5.18	6.50	.79	.60	.14	6.07
13	.31	21.28	10.28	11.81	14.03	3.62	3.27	1.69	2.98	.30	.14	.16	5.82
14	.81	24.73	35.22	9.60	5.98	6.89	5.47	13.43	1.56	.50	.42	.09	8.72
15	1.66	10.40	10.52	9.75	13.93	3.27	7.27	5.90	2.55	.78	.38	.05	5.54
16	.48	10.89	21.56	5.61	13.94	5.30	29.14	5.40	1.63	.43	.65	.13	7.93
17	.57	3.62	9.20	7.90	10.78	4.14	2.30	7.42	2.70	.53	.33	.17	4.14
18	.81	3.79	10.00	7.26	6.36	3.20	3.61	5.41	3.10	.24	.10	.08	3.66
19	.50	2.97	9.20	13.32	9.51	3.67	4.59	4.07	2.99	.24	.17	.14	4.28
20	.35	4.37	33.80	23.59	9.04	5.98	4.35	2.46	1.40	.43	.09	.06	7.16
21	.54	22.46	21.52	8.29	7.62	4.24	2.20	2.40	2.41	.14	.38	.09	6.03
22	.81	8.42	51.00	6.27	9.99	4.93	15.83	3.24	1.93	.18	.10	.16	8.57
23	.63	4.01	18.97	13.54	5.13	1.87	4.75	2.00	.89	.11	.14	.07	4.34
24	.93	2.91	7.37	6.85	10.95	6.98	5.30	2.12	1.94	.29	.23	.06	3.83
25	.34	3.60	10.30	3.62	7.08	3.21	2.55	1.71	2.43	.15	.15	.13	2.94
26	.50	10.24	21.97	25.73	38.55	2.44	5.46	3.33	2.78	.70	1.16	.40	9.44
27	2.33	6.44	44.69	23.05	15.44	6.49	4.32	1.66	2.57	.61	.76	.13	9.04
28	.46	3.79	8.60	15.86	9.14	1.67	6.08	5.92	2.08	.25	.87	.04	4.56
29	.85	5.93	11.40	14.21	4.13	2.19	4.30	3.62	1.44	.17	.33	.05	4.05
30	.13	2.59	24.31	15.80	3.73	7.33	3.91	3.04	1.64	.90	.75	.15	5.36
31	.49	.70	4.01	7.33	5.69	9.48	8.93	8.13	4.66	.54	1.09	.30	4.28
MEAN	.71	6.92	16.17	14.34	11.67	5.75	6.40	4.33	2.83	.34	.36	.11	5.83
S D	.51	6.16	11.56	10.18	8.16	4.55	5.11	2.54	1.54	.22	.30	.08	1.93

TABLE 7

(continued....)

ESTIMATED MEAN MONTHLY DISCHARGE(M3/S)

DIYABANA OYA

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	
1	.30	.51	1.19	.28	1.35	.15	.05	.10	.04	.04	.23	.06	.36
2	.12	.24	.33	.92	.44	.42	.14	.08	.09	.07	.09	.04	.25
3	.09	.42	1.03	1.15	.46	.28	.29	.38	.09	.04	.38	.05	.39
4	.04	.04	.33	.10	.46	.11	.04	.14	.18	.18	.08	.09	.15
5	.13	.12	.79	1.48	.10	.16	.23	.22	.13	.07	.06	.04	.30
6	.08	.17	.47	.20	1.09	.75	.10	.16	.08	.03	.03	.07	.27
7	.06	.18	1.44	.84	.95	.47	.28	.10	.27	.06	.02	.12	.40
8	.12	.42	1.01	1.44	.75	.13	.34	.37	.31	.09	.06	.02	.42
9	.28	.22	.46	.27	.11	.14	.17	.07	.04	.05	.10	.10	.17
10	.11	.33	.44	.45	.99	.87	.51	.44	.36	.07	.10	.04	.39
11	.08	.13	.63	.50	.59	.32	.17	.45	.43	.05	.06	.07	.29
12	.17	.23	.39	.75	.65	.33	.20	.23	.16	.15	.06	.03	.28
13	.06	.37	.70	.43	.72	.76	.41	.31	.16	.08	.07	.08	.35
14	.10	.65	5.35	3.53	1.08	.46	.15	.24	.05	.14	.03	.04	.99
15	.08	.51	.67	.58	.48	.11	.14	.18	.10	.06	.06	.07	.25
16	.18	.42	.64	.95	.90	.48	.07	.06	.06	.06	.06	.08	.33
17	.14	1.59	1.35	.73	.49	.47	.13	.16	.11	.09	.13	.07	.46
18	.16	.15	1.96	.95	.79	.22	.53	.45	.06	.04	.02	.07	.45
19	.08	.51	.40	.65	.31	.18	.21	.22	.06	.02	.04	.11	.23
20	.16	.25	.52	.30	.83	.62	.27	.42	.05	.09	.12	.17	.32
21	.11	.23	.95	1.00	1.43	.64	.25	.27	.05	.03	.03	.04	.42
22	.04	.47	1.08	1.13	.65	.79	.82	.17	.03	.04	.13	.06	.45
23	.13	.38	.62	.57	.38	.47	.10	.22	.15	.14	.03	.05	.27
24	.09	.43	2.56	1.25	1.50	.35	.11	.18	.07	.05	.12	.16	.57
25	.09	.55	.42	.30	.30	.60	.39	.28	.14	.21	.10	.08	.29
26	.05	.24	1.02	1.13	1.04	.10	.16	.19	.19	.04	.06	.07	.36
27	.36	.38	.34	.94	.89	.29	.26	.32	.10	.08	.14	.10	.35
28	.09	.22	1.26	1.33	1.09	.40	.35	.70	.09	.10	.17	.05	.49
29	.17	1.15	2.17	1.08	1.12	.13	.03	.29	.07	.04	.03	.02	.52
30	.06	.77	1.01	.98	.57	.58	.29	.32	.16	.03	.04	.05	.40
31	.16	.24	.32	.74	.45	.37	.26	.34	.32	.05	.19	.22	.30
MEAN	.13	.40	1.03	.87	.74	.39	.24	.26	.14	.07	.09	.07	.37
S D	.08	.31	.98	.62	.37	.23	.17	.14	.10	.05	.07	.04	.15

TABLE 7

(continued....)

ESTIMATED MEAN MONTHLY DISCHARGE(M3/S)

HEPOLA OYA

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	
1	.48	3.09	9.01	2.32	1.38	2.19	1.22	1.05	.46	.49	.72	.34	1.89
2	.22	.55	6.82	4.26	7.23	3.81	3.36	1.29	.76	.68	.33	.48	2.48
3	.75	4.52	13.86	14.30	12.52	4.77	3.20	2.43	.36	.30	1.16	.21	4.85
4	.65	.35	7.29	3.16	1.88	.83	.57	2.37	.21	.16	.19	.74	1.53
5	.63	1.04	4.57	11.39	3.08	1.68	1.01	.79	.63	.54	.52	1.00	2.24
6	.82	2.93	7.33	1.49	2.39	3.05	.32	1.44	.83	.78	.33	.37	1.84
7	.46	2.55	5.27	6.83	6.75	1.64	3.84	1.21	1.62	.22	.08	1.13	2.63
8	3.18	4.84	6.62	5.25	3.99	2.57	1.88	6.84	1.20	.63	.39	.60	3.17
9	1.58	3.60	4.72	3.79	6.75	3.54	1.64	2.50	.50	.31	.21	.41	2.46
10	1.02	4.51	4.36	5.61	3.13	6.55	3.26	1.79	.72	.49	.65	.25	2.70
11	.95	.78	7.13	7.22	17.04	5.53	2.79	4.09	4.30	.52	.29	.75	4.28
12	.61	3.69	5.73	5.70	7.01	2.59	1.33	1.81	4.88	.58	.38	.87	2.93
13	2.29	3.36	7.46	9.14	8.72	3.35	.94	.60	.89	2.56	1.67	.58	3.47
14	.81	1.91	9.85	4.41	4.41	3.56	1.18	4.98	1.15	.89	1.34	1.92	3.03
15	.60	2.08	6.99	4.94	1.57	2.14	1.42	1.17	.63	1.27	.14	.41	1.95
16	.97	8.71	13.14	6.07	8.97	1.07	2.77	2.96	1.46	.65	.67	.26	3.98
17	.74	6.42	1.60	3.09	12.00	2.06	1.09	2.55	.88	.51	.50	.26	2.64
18	.23	.82	3.18	8.42	3.11	1.02	1.90	1.97	.73	.54	.26	.60	1.90
19	1.32	3.74	9.73	7.69	5.83	.99	1.05	2.53	.45	1.95	.59	.23	3.01
20	1.84	5.29	6.44	17.12	5.31	2.99	.87	.62	.50	.21	.78	.75	3.56
21	.56	2.09	2.21	2.69	2.18	.23	.56	3.57	.59	.20	.48	.52	1.32
22	.25	1.67	3.11	3.38	3.23	3.30	3.34	3.11	.50	.40	.19	.40	1.91
23	.89	3.43	3.27	6.20	7.55	3.06	.86	2.45	.57	.63	.16	.19	2.44
24	1.51	1.73	3.41	4.61	2.41	8.49	1.59	1.32	2.43	.98	1.11	.64	2.52
25	1.42	2.13	2.53	8.01	3.99	2.56	2.64	1.18	1.64	.56	.32	.80	2.31
26	3.45	1.81	4.05	8.20	10.81	1.73	2.17	.96	.46	.79	.64	.37	2.95
27	.78	2.30	9.04	2.28	4.54	4.81	4.33	1.53	1.21	.98	3.80	1.43	3.09
28	1.36	1.30	11.54	5.84	3.86	.76	1.37	3.87	.93	.69	.34	.34	2.68
29	1.52	2.60	8.25	3.91	4.22	1.04	.80	1.50	.70	.61	1.17	.91	2.27
30	.28	3.04	5.66	2.35	3.75	1.36	1.30	1.68	1.38	.59	.84	1.06	1.94
31	1.78	3.50	4.82	9.89	2.99	6.13	4.29	1.13	.85	.36	1.02	.53	3.11
MEAN	1.10	2.92	6.42	6.11	5.57	2.88	1.90	2.17	1.11	.68	.69	.63	2.68
S D	.79	1.81	3.10	3.56	3.68	1.92	1.16	1.38	1.04	.50	.70	.39	.79

TABLE 7

(continued....)

ESTIMATED MEAN MONTHLY DISCHARGE(M3/S)

LOGGAL OYA

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	AV
1	2.01	7.53	64.67	8.86	7.55	1.97	2.77	2.43	1.04	.53	1.36	.59	8.44
2	2.31	3.66	19.71	10.11	6.30	7.18	4.55	4.74	1.75	.78	.46	.74	5.19
3	2.04	5.79	9.43	12.14	5.86	8.46	1.14	6.43	.83	.84	3.12	.86	4.75
4	.45	1.06	3.27	2.91	8.30	3.16	2.45	7.08	2.84	1.72	.39	.17	2.82
5	.38	1.40	9.50	16.32	7.65	1.89	1.21	2.90	2.52	1.27	.97	.80	3.90
6	.38	7.55	13.99	8.67	3.67	6.81	.77	3.95	1.56	1.09	1.38	2.46	4.35
7	3.24	2.13	13.35	71.92	12.25	4.87	2.10	2.51	1.18	1.09	.22	.32	9.60
8	2.43	3.67	19.83	13.77	7.58	1.95	3.89	13.92	3.68	2.56	2.08	.69	6.34
9	1.49	.62	2.91	4.47	5.28	3.45	2.22	3.22	.97	1.37	1.15	.95	2.34
10	1.09	1.64	4.76	11.74	12.14	7.99	6.20	4.55	1.90	.33	.92	.76	4.50
11	5.78	18.34	18.01	27.51	9.41	4.71	9.39	5.99	2.13	2.74	1.03	.44	8.79
12	1.33	3.82	5.75	2.74	7.13	3.62	1.16	2.16	2.00	.93	.77	1.51	2.74
13	1.91	4.63	6.67	12.02	27.05	13.66	3.78	1.20	1.48	8.77	.81	.76	7.06
14	3.61	16.63	29.07	17.10	13.59	14.19	4.82	4.31	1.87	2.27	1.33	.55	9.11
15	2.21	11.74	3.14	5.61	12.29	2.45	1.90	2.42	2.00	.57	1.10	1.09	3.88
16	1.21	2.20	8.27	4.39	22.28	13.87	4.44	4.44	3.09	1.80	1.17	1.19	5.70
17	1.11	6.09	11.46	16.13	11.95	5.24	1.53	4.67	1.54	1.34	2.76	2.12	5.50
18	2.11	8.24	23.26	15.79	4.19	2.22	3.09	7.61	.76	.81	.77	1.46	5.86
19	1.34	5.98	9.01	8.37	4.70	2.13	3.61	3.80	2.52	1.62	.35	.51	3.66
20	1.37	2.84	8.92	4.55	5.28	5.41	2.06	2.02	1.99	1.26	.97	3.35	3.34
21	.99	5.70	9.60	4.41	11.08	3.82	3.79	6.28	1.71	.98	1.36	1.58	4.28
22	1.23	6.47	14.82	4.26	6.70	2.66	5.35	1.52	.90	1.16	.62	2.39	4.01
23	1.63	3.99	5.39	5.13	8.38	3.14	12.05	8.71	.76	.58	.18	.73	4.22
24	3.57	14.36	27.07	23.25	29.20	13.62	3.26	4.00	.87	1.60	2.51	1.05	10.36
25	.93	3.21	13.55	16.07	17.78	10.17	3.33	5.49	1.54	.53	.38	.34	6.11
26	1.07	6.37	5.20	8.29	8.39	1.97	3.53	1.64	.55	.41	.82	.55	3.23
27	1.38	3.74	50.95	62.67	4.68	4.14	1.86	1.15	1.89	.73	1.21	2.39	11.40
28	2.94	3.11	22.12	8.57	14.83	5.31	2.47	3.70	.77	.59	1.19	1.00	5.55
29	3.30	13.89	9.63	12.40	18.76	8.46	3.26	2.95	2.98	.70	.93	.80	6.50
30	.58	3.87	11.80	17.55	7.40	15.12	5.70	14.13	4.73	2.92	2.84	1.84	7.37
31	2.25	6.40	9.35	9.89	7.15	2.84	5.03	3.08	2.26	1.17	2.01	1.68	4.43
MEAN	1.86	6.02	15.05	14.44	10.61	6.02	3.64	4.61	1.83	1.45	1.20	1.15	5.66
S D	1.17	4.55	13.44	15.36	6.47	4.22	2.39	3.16	.95	1.52	.77	.76	2.37

Thus, the results and inferences drawn from a reservoir operation model that uses this one set of data might be subject to a bias that occurred entirely by chance. This danger can be reduced by using a number of sequences of synthetic data and assessing the sensitivity of the operation model to the different sequences. Provided little variation is apparent in the output data, subsequent operational analysis may be restricted to a single sequence.

The flow generation model is designed to preserve the mean and variance as well as the correlation structure of the historic data. Generally, the preservation of these features in the synthetic data will be sufficient to allow the operation of a reservoir system to be examined realistically. There may, however, be other statistics to which given aspects of reservoir operation may be particularly sensitive. For example, the timing of gate closure for filling a reservoir towards the end of a wet season may be extremely sensitive to the rate of recession. Should such unusual sensistivity become apparent during the early testing of the operation model, consideration should be given to increasing the sophistication of the flow generation model to correct such inadequacies.

FLOOD ANALYSIS

Flood flow data for the Mahaweli Ganga catchment were only available for a limited number of sites with record lengths varying between 5 and 33 years, thus precluding the use of statistical methods for estimating spillway design floods. The raingauge network is very dense over most of the catchment and has been in operation since about 1880 and thus provides reliable daily data over a long period. We believe that the most reliable estimate of the spillway design flood, using the available data, follows from the conversion of the probable maximum precipitation (PMP) to the probable maximum flood (PMF) using the unit hydrograph/losses model.

Because the discharge data at Victoria were considered to be unreliable, they were not used in any of the previous analyses. However, good data were available for Gurudeniya, about 15 km upstream of the dam site, and had already been used for the flow record extension; these data, together with the available catchment rainfall data, were used to derive a unit hydrograph for Gurudeniya. It was decided to apply this unit hydrograph to the Victoria dam site catchment.

Rainfall data were available for over twenty large storms which covered the whole of the upper Mahaweli catchment. A simplified moisture maximisation technique was therefore used to estimate the PMP over the catchment from the data of the largest storm ever recorded. The unit hydrograph/losses model was then used to convert the PMP to the PMF.

As very few rainfall and discharge data were available for the Ulhitiya catchment, a synthetic unit hydrograph was derived using the results from a local study (8) and also the Floods Study Report (FSR) (9). The design storm was based on a PMP calculated using Hershfield's statistical approach, and a storm profile based on very limited local data. The PMF was then calculated from the synthetic unit hydrograph and the estimated PMP.

A regional flood frequency curve has been drawn from the observed flood data at ten sites in or near the study area. Once the mean annual flood, $\bar{Q}$, has been determined from observed records, or from the catchment characteristics, annual floods of return periods up to about 50 years can be deduced from the curve. For construction purposes, the series of observed monthly maximum daily discharges at two sites were used to investigate the magnitude of short return period monthly and seasonal floods.

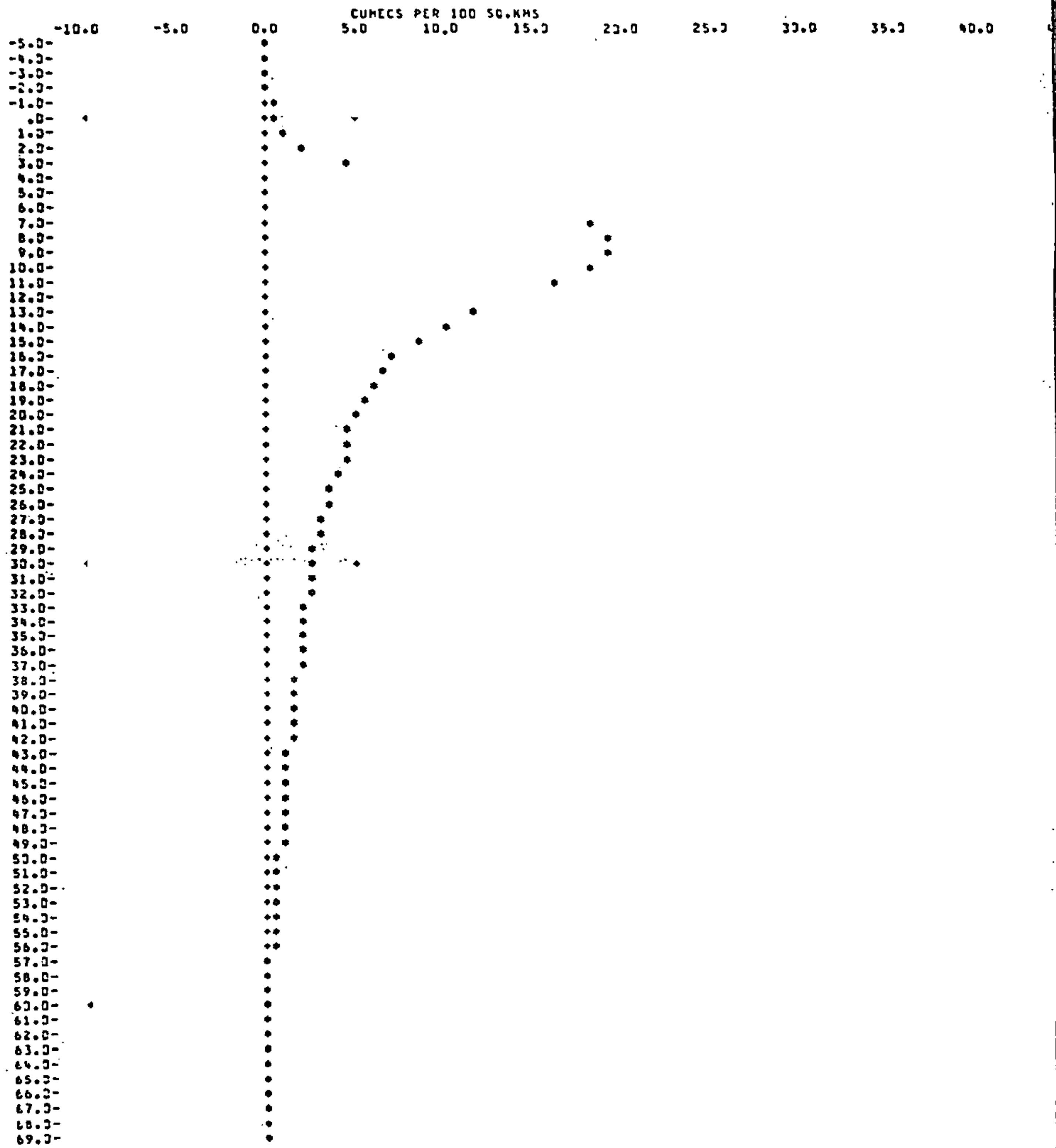
Derivation of the unit hydrographs

The hourly stage readings from Gurudeniya were examined and a number of well defined flood events chosen for hydrograph analysis. On examination of the daily rainfall data corresponding to these events, it appeared that there were only two storms in which both the flood and rainfall data were suitable.

The catchment rainfall for the storms of 15-17 July 1972 and 26 May 1962 were calculated and transformed to hourly data using the storm profiles recorded at Kandy and Nuwara Eliya. These data and the corresponding flood discharge hydrographs were used as the input data to a unit hydrograph separation program that is described in the FSR. An example of the output from the program is given in Table 8, which refers to the 1962 flood and shows the derived unit hydrograph. The derived hydrograph from the 1972 storm was not satisfactory as there appeared to be some unexplainable discrepancies between the rainfall and discharge data. The unit hydrograph has therefore been based on the results from the 1962 storm.

The unit hydrograph thus derived was then tested by using the rainfall data from a storm in August 1947 to predict the peak discharge of the corresponding flood at Gurudeniya. The predicted peak of 5715 m^3/s (201 800 ft^3/s) is consistent with the observed peak of 5947 m^3/s (210 000 ft^3/s). A further test involved using the unit hydrograph with the rainfall of a storm in 1972, calculated over the Victoria gauging station catchment, to predict the corresponding flood hydrograph. Although the complete flood hydrograph was not recorded at Victoria, as observations have often been restricted to daylight hours alone, sufficient points exist for a comparison to

GURUDENIYA : DERIVED UNIT HYDROGRAPH



be made. In Figure 9 the observed and predicted hydrographs are plotted and are comparable in terms of peak discharge and shape. The predicted peak of the hydrograph is lower than the corresponding recorded peak at Gurudeniya, but this may be explained by attenuation in the reach between the two stations.

These two tests give no cause for rejecting the derived unit hydrograph, and we believe that it can be used to predict high return period floods at the Victoria dam site, when used with the storm rainfall calculated for the whole catchment.

At Ulhitiya, insufficient data were available to deduce a unit hydrograph in the conventional way; a synthetic unit hydrograph was derived from catchment characteristics using empirical relationships from Snyder (11) and the FSR (9) to estimate the hydrograph time to peak, T_p

Snyder's formula gives T_p as

$$T_p = C_t(L.L_c)^{0.3}$$

where L is the length from the catchment outlet to the catchment boundary,

L_c is the length from the outlet to the nearest point on the river to the centroid of the catchment,

and C_t is a coefficient.

Here a value of C_t was taken from (8) and the equation gives a time to peak of 12.28 hours.

The following regression equations for T_p were derived in the FSR:

$$T_p = 20.46\ S^{-0.598}$$

and $$T_p = 2.8\ \left(\frac{L}{\sqrt{S}}\right)^{0.47}$$

where T_p is the time to peak in hours,

L is the length of the main stream channel from the catchment outlet in km,

and S is the slope between the 10 and 85 percentiles of L in m/km.

OBSERVED & PREDICTED HYDROGRAPH AT VICTORIA

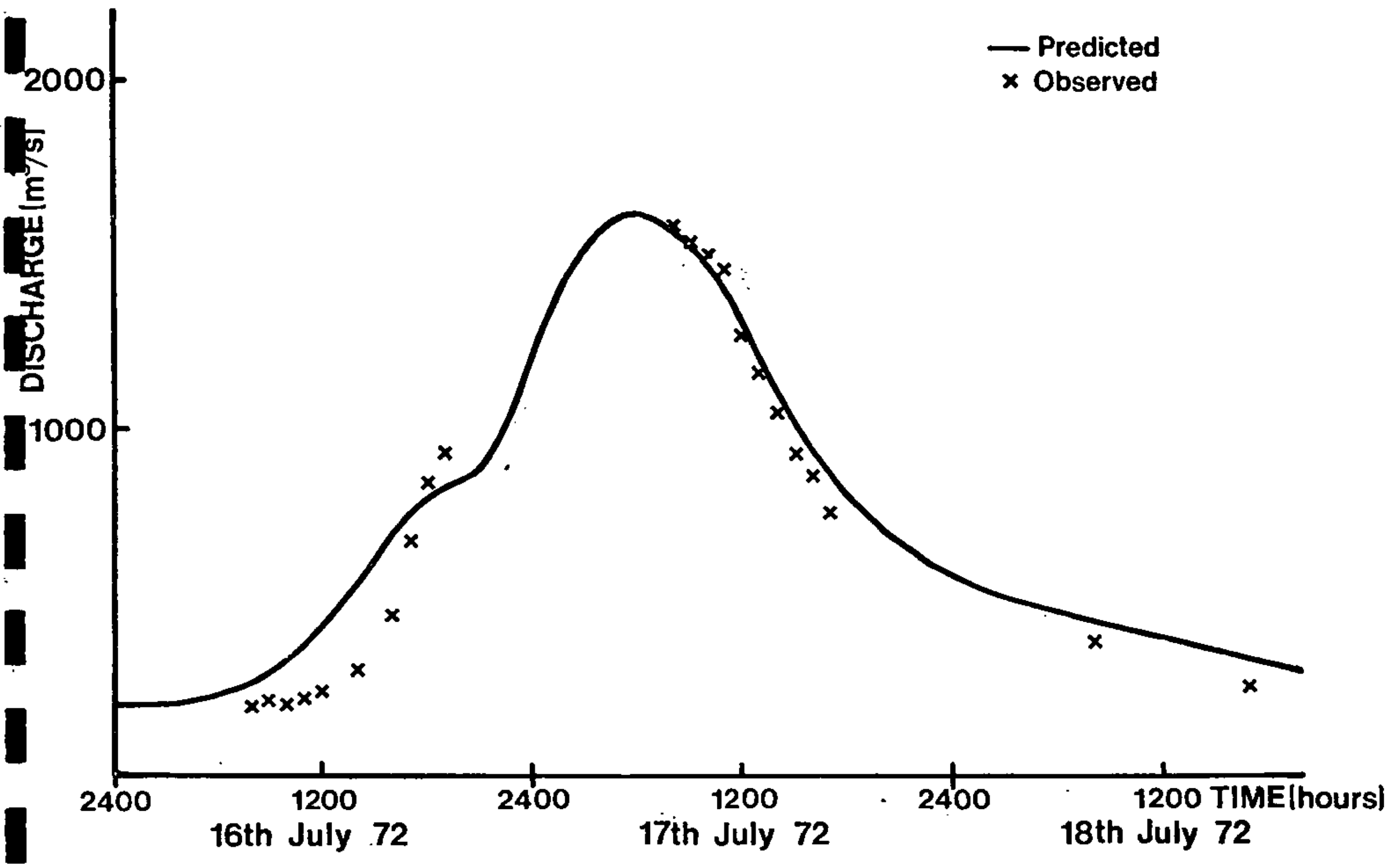


Figure 9

Using these equations, the predicted times to peak are 11.73 and 10.78 hours respectively.

The mean of these three values, 11.60, was used to define the synthetic triangular unit hydrograph shown in Figure 10, from the following equations derived in the FSR:

$$Q_p = \frac{220}{T_p} \quad m^3/s/100\ km^2$$

and $T_B \doteq 2.52\ T_p$ hours

where Q_p is the peak discharge in m^3/s,
and T_B is the time base of the hydrograph in hours.

Analysis of extreme rainfall

Estimation of the PMP by transposition of storms was not considered for this study due to the unique topographic and climatic conditions of the upper Mahaweli catchment. Instead the largest recorded storm was analysed and used to estimate the PMP by a simplified method of moisture maximization. Much of the data for the following analysis of moisture maximization has been taken from an unpublished paper (12). From a study of the isohyetal map of the period corresponding to the south-west monsoon, it was realised that the heaviest rainfall is generally recorded at Watawala. It was therefore assumed that the daily rainfall at this raingauge could be used to indicate the occurrence of heavy rainfall within the catchment. Days on which more than 5 inches of rain fell were considered as storms, and during the period 1911 to 1971 there were 190 such events.

The corresponding falls at 5 other stations, namely Kandy, Hatton, Nuwara Eliya, Watagoda and New Forest, were then extracted from the daily records. The mean of the falls at the six stations was then taken as an index of the catchment rainfall, in order to select the 30 heaviest storms for further analysis. Isohyetal maps for each storm were then drawn using the daily data from 32 raingauges

SYNTHETIC UNIT HYDROGRAPH FOR ULHITIYA

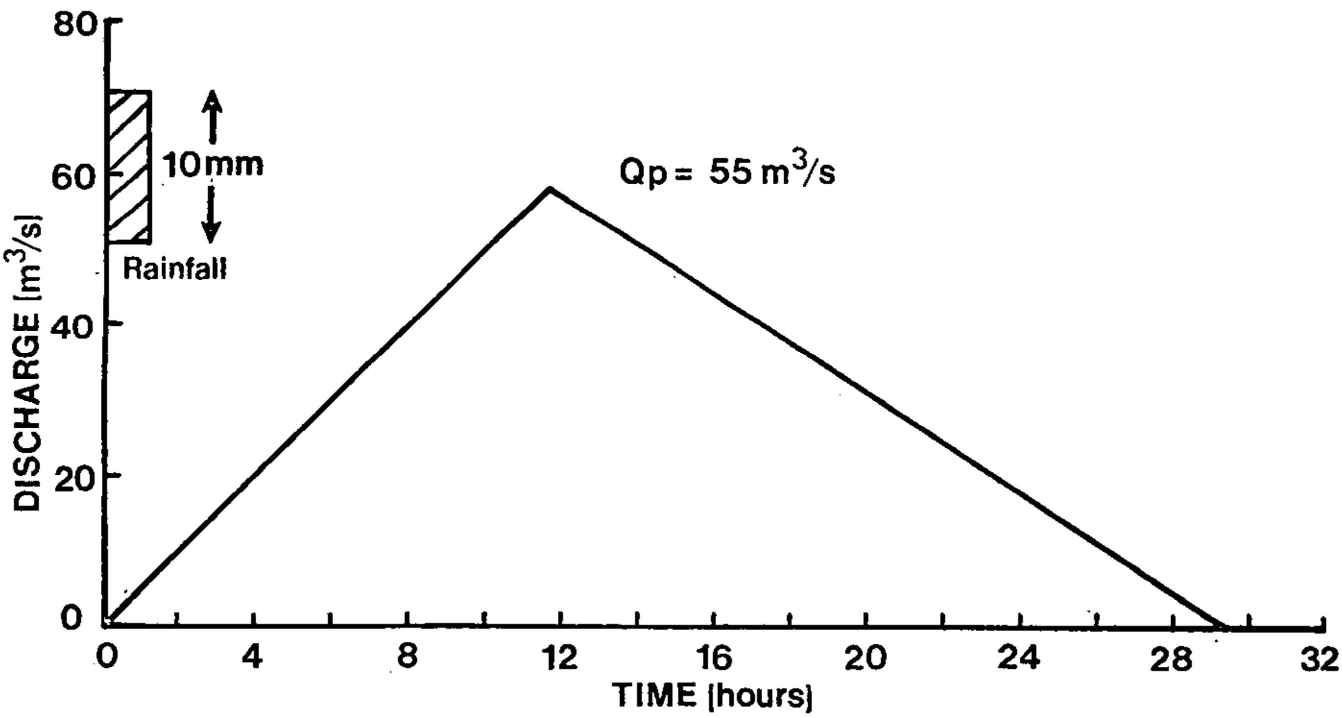


Figure 10

within the catchment. The August 1947 storm was the heaviest of these storms; the extent of the storm can be seen in Figure 6, and the isohyets are shown in Figure 11.

It was decided to use the 1947 storm to estimate the PMP at Victoria as it was widespread over the catchments, it was the heaviest storm on record, and it produced the largest recorded floods on the upper Mahaweli catchments. The storm was therefore the closest recorded storm to the PMP. Data from the climatological stations at Kandy and Nuwara Eliya were used for the analysis of moisture maximization which is described briefly.

Generally, a storm can be maximised with respect to wind speed, storm efficiency or moisture content, but because of the extreme nature of the 1947 storm, we have assumed its wind speed and efficiency to be typical of the maximum possible. The problem was therefore reduced to one of maximising the moisture content alone.

The method of moisture maximisation assumes that the ratio of the maximum dew point ever recorded to the dew point recorded in a given storm is equal to the ratio of the maximum ever precipitable water to the precipitable water of that storm. The rainfall for an extreme storm can then be multiplied by the corresponding dew point ratio to give an estimate of the PMP. This method, which is described in more detail in the Manual for the Estimation of PMP (13) also relies on nomograms from Wiesner (14).

A more rigorous method often used for calculating the PMP for orographic rainfall involves the use of a dynamic meteorological model that requires climatological and windspeed data at different altitudes. Such data were not available, so this approach was not attempted. The storm maximisation outlined above however gives a reasonable, albeit low, estimate of the PMP as no maximization of the storm efficiency or windspeed has been attempted.

ISOHYETAL MAP FOR AUGUST 1947 STORM [inches]

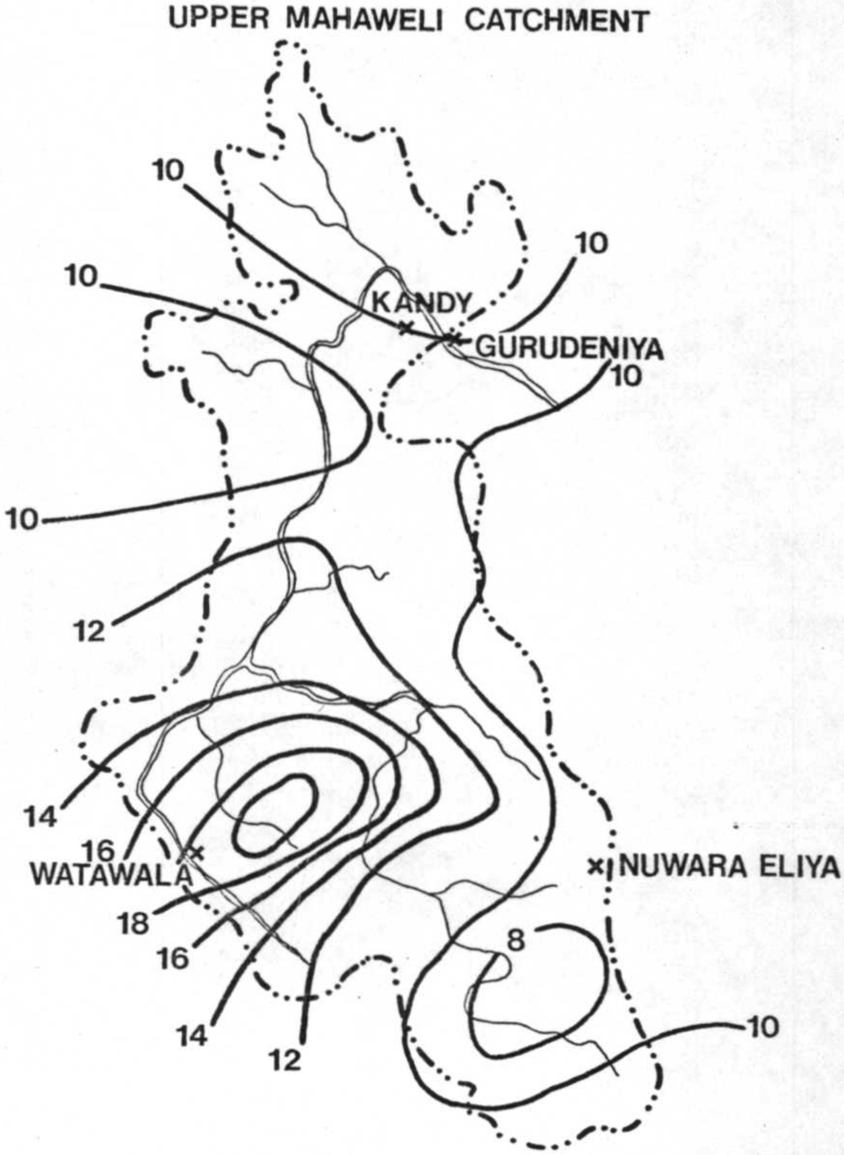


Figure 11

The PMP over the catchment to the Victoria dam was calculated from the isohyets of the August 1947 storm multiplied by a maximising ratio determined using data from Kandy and Nuwara Eliya, shown in Table 9.

Hershfield's (10) statistical method was also used to estimate the 1-day PMP, and provides an upper bound estimation from the equation

$$\text{1-day PMP} = \bar{x} + K_t \sigma$$

where $\bar{x}$ is the mean of the series of annual maximum daily rainfall at a given station,

σ is the standard deviation of the series,

and K_t is a constant, often taken as 15.

Two long-term (61 years) series of data were available for Kandy and Nuwara Eliya, and were used with a value of K_t = 15 to give estimates of 640 mm and 678 mm respectively at the two stations. These results are compared with the results of moisture maximisation in Table 9.

An estimate of the 1000 year storm at Victoria was made from the 1000 year 1-day rainfall for the Gurudeniya catchment, which had been calculated previously from statistical analysis of the rainfall data (12). The corresponding rainfall over the dam site catchment was estimated using the ratio of the long-term mean annual catchment rainfalls. We consider that the 1-day 1000 year rainfall would underestimate the critical 1000 year storm for the catchment; we have therefore assumed that a storm duration of 2 days would be more realistic for deriving the 1000 year flood. An estimate of the 2-day catchment rainfall was made consequently based on storms recorded over two days.

Flooding on the Ulhitiya Oya occurs largely during the north-east monsoon, thus maximization of the 1947 storm which resulted from

TABLE 9

ESTIMATES OF THE 1-DAY PMP

Station	Maximum recorded rainfall		PMP Storm maximization		PMP Hershfield	
	(mm)	(in)	(mm)	(in)	(mm)	(in)
Kandy	441	17.38	525	20.68	640	25.18
Nuwara Eliya	245	9.64	312	12.30	678	26.71

the south-west monsoon would be inappropriate. Moreover as the rain-gauge network in this region is sparse, and adequate climatological data are not available, the technique of storm maximization could not be used.

Instead, the 42 year records of daily rainfall from Horaborawewa and Bibile, which are close to the Ulhitiya catchment, were used to estimate the 1-day PMP by Hershfield's method. Taking a value of $K_t = 15$ (see above), the point 1-day PMP for Ulhitiya was taken as the mean of the estimates at the two stations, and gives a 1-day PMP of 732 mm. To calculate the 1-day areal PMP, this number has to be multiplied by an areal reduction factor. Insufficient data were available to allow this factor to be calculated. However, experience in other parts of the world suggested that a value of 0.8 would be appropriate.

Design rainfall

A choice of duration and time distribution or profile of the rainfall must be made to specify the design storm. At Victoria the catchment PMP has been estimated from an observed storm. The profile of the PMP was consequently based on the recorded profile of the 1947 storm at Nuwara Eliya, as these were the only short duration data available for that time. The duration of the storm was similarly fixed as the duration of the 1947 storm. Similar assumptions were made for the profile of the 1000 year storm.

For Ulhitiya, we have had to make other simplifying assumptions, concerning the design storm. In the absence of local information comparing the 1-day and 2-day falls for various return periods, a single storm duration of 24 hours was chosen. A storm profile was deduced from the available depth-duration-frequency curves (2), and combined with an areal reduction factor of 0.8 to define the design storm.

Unit hydrograph convolution program

A program described in the FSR calculates and prints out the convolution of net rainfall and unit hydrograph ordinates for a given design storm and unit hydrograph under a variety of conditions. These conditions typically concern baseflow, the wetness of the catchment at the start of the storm and the percentage runoff during the storm; they are referenced in the program as ANSF (average non-separated flow), CWI (catchment wetness index) and SPR (standard percentage runoff) respectively.

ANSF was determined from a long recession that was recorded immediately before a large flood. For the analysis of extreme floods, it is usually assumed that the catchment is wet at the start of the storm; CWI has been chosen at a conservative value with this in mind. The percentage runoff is also chosen to reflect the extreme nature of the events under consideration, and is usually deduced from observed data.

Spillway design floods

The two design storms discussed above were used as input data to the convolution program to estimate the probable maximum flood from the probable maximum precipitation.

At Victoria, a percentage runoff of 70 per cent was used, compared with the observed 56 per cent in August 1947, and produced the computer output given in Table 10; the hydrograph is illustrated in Figure 12. The peak discharge is estimated as 9510 m^3/s (335,800 ft^3/s) and the hydrograph has a time base of 141 hours. Although the time base is long, the characteristics of the storm and the flood hydrograph (ie, a long time base and multiple peaks) closely resembles the large storms that tend to occur during the south-west monsoon.

At Ulhitiya the flood peak was 4160 m^3/s (about 146,900 ft^3/s) with a time base of 47 hours. The output is given in Table 11, and the resulting hydrograph illustrated in Figure 13.

PROBABLE MAXIMUM FLOOD AT VICTORIA

APPLICATION OF PROBABLE MAXIMUM PRECIPITATION TO UNIT HYDROS

VICTORIA PROBABLE MAXIMUM FLOOD.

PERCENTAGE RUNOFF INCREASING THROUGH STORM WITH CWI

AREA (SQ.KM.)	1891.0
DATA INTERVAL (HR)	1.0
DESIGN DURATION (HR)	75.0
TOTAL RAIN (MM)	766.5
PERCENTAGE RUNOFF	70.1
ANSF (CUMECS PER SQ.KM.)	.1370
CWI AT START OF STORM	200.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH
.00	21.64	7.48	.01	259.33
1.00	23.23	8.86	.02	259.57
2.00	21.70	8.98	.07	260.64
3.00	11.80	5.05	.16	263.06
4.00	5.52	2.38	.43	269.43
5.00	3.99	1.72	.63	278.83
6.00	2.28	.98	1.19	295.77
7.00	3.99	1.72	1.89	321.69
8.00	.00	.00	4.61	384.90
9.00	.76	.32	8.12	499.55
10.00	.76	.32	12.89	682.74
11.00	2.29	.95	15.85	896.40
12.00	3.04	1.26	18.18	1109.89
13.00	5.90	2.46	18.87	1292.71
14.00	8.56	3.65	19.08	1406.00
15.00	11.04	4.65	17.96	1473.70
16.00	11.42	5.17	16.15	1482.66
17.00	13.71	6.43	13.95	1442.35
18.00	8.56	4.08	11.73	1359.98
19.00	1.52	.72	9.87	1265.84
20.00	40.74	21.74	8.27	1175.15
21.00	28.93	16.61	7.21	1112.45
22.00	30.64	18.87	6.35	1086.26
23.00	31.41	20.67	5.79	1109.82
24.00	31.40	21.95	5.34	1186.17
25.00	77.66	63.49	5.01	1307.93
26.00	42.64	37.23	4.72	1468.75
27.00	11.61	10.15	4.50	1649.75
28.00	27.79	25.09	4.29	1906.38
29.00	33.31	31.31	4.04	2250.37
30.00	21.70	20.75	3.70	2721.68
31.00	9.90	9.43	3.34	3276.63
32.00	19.42	18.73	3.03	3933.34
33.00	4.57	4.34	2.81	4770.72

TABLE 10

(continued....)

34.00	3.62	3.39	2.63	5757.77
35.00	4.76	4.40	2.48	6604.31
36.00	1.90	1.72	2.36	7696.36
37.00	2.28	2.03	2.26	8429.18
38.00	6.47	5.71	2.17	8993.80
39.00	6.47	5.66	2.08	9369.13
40.00	3.62	3.12	1.97	9510.18
41.00	6.09	5.20	1.88	9382.00
42.00	4.19	3.53	1.79	9075.54
43.00	12.18	10.30	1.71	8597.77
44.00	8.38	7.06	1.61	8038.18
45.00	8.75	7.35	1.51	7416.72
46.00	6.47	5.39	1.40	6823.75
47.00	7.43	6.15	1.32	6282.66
48.00	3.81	3.11	1.25	5821.30
49.00	2.86	2.30	1.18	5452.44
50.00	4.19	3.33	1.11	5156.17
51.00	2.86	2.24	1.05	4956.97
52.00	4.76	3.69	.98	4824.13
53.00	11.61	9.05	.89	4757.54
54.00	4.76	3.67	.80	4717.59
55.00	4.57	3.49	.71	4673.12
56.00	2.86	2.15	.61	4598.95
57.00	1.90	1.41	.53	4511.05
58.00	.57	.41	.45	4387.97
59.00	.57	.41	.40	4241.69
60.00	4.57	3.23	.35	4080.13
61.00	5.71	4.01	.30	3935.60
62.00	2.28	1.58	.24	3612.65
63.00	2.67	1.82	.18	3714.86
64.00	5.52	3.75	.12	3614.66
65.00	2.86	1.92	.05	3511.78
66.00	4.19	2.79	.02	3393.11
67.00	11.04	7.41	.00	3265.36
68.00	5.52	3.69		3128.09
69.00	6.47	4.31		2994.68
70.00	1.90	1.25		2673.57
71.00	1.33	.86		2765.70
72.00	.57	.36		2673.31
73.00	7.80	4.96		2597.87
74.00	2.67	1.68		2542.24
75.00				2512.06
76.00				2499.59
77.00				2505.01
78.00				2492.76
79.00				2465.01
80.00				2401.14
81.00				2330.69
82.00				2241.09
83.00				2144.51
84.00				2022.54
85.00				1888.18
86.00				1745.53
87.00				1607.54
88.00				1475.77
89.00				1350.37
90.00				1235.45

91.00	1132.42
92.00	1044.78
93.00	969.41
94.00	906.23
95.00	851.61
96.00	804.74
97.00	762.00
98.00	723.08
99.00	686.95
100.00	654.62
101.00	625.37
102.00	598.83
103.00	573.43
104.00	549.47
105.00	527.12
106.00	506.44
107.00	487.04
108.00	466.70
109.00	451.69
110.00	436.31
111.00	422.26
112.00	409.53
113.00	397.47
114.00	386.38
115.00	375.69
116.00	366.04
117.00	356.39
118.00	347.07
119.00	338.46
120.00	330.65
121.00	323.52
122.00	316.77
123.00	310.36
124.00	304.27
125.00	298.44
126.00	293.09
127.00	288.11
128.00	283.66
129.00	279.38
130.00	275.47
131.00	271.85
132.00	268.60
133.00	256.33
134.00	264.40
135.00	262.97
136.00	261.61
137.00	260.88
138.00	260.05
139.00	259.49
140.00	259.22
141.00	259.16

CURVATURE AROUND PEAK = -269.224

PROBABLE MAXIMUM FLOOD AT VICTORIA

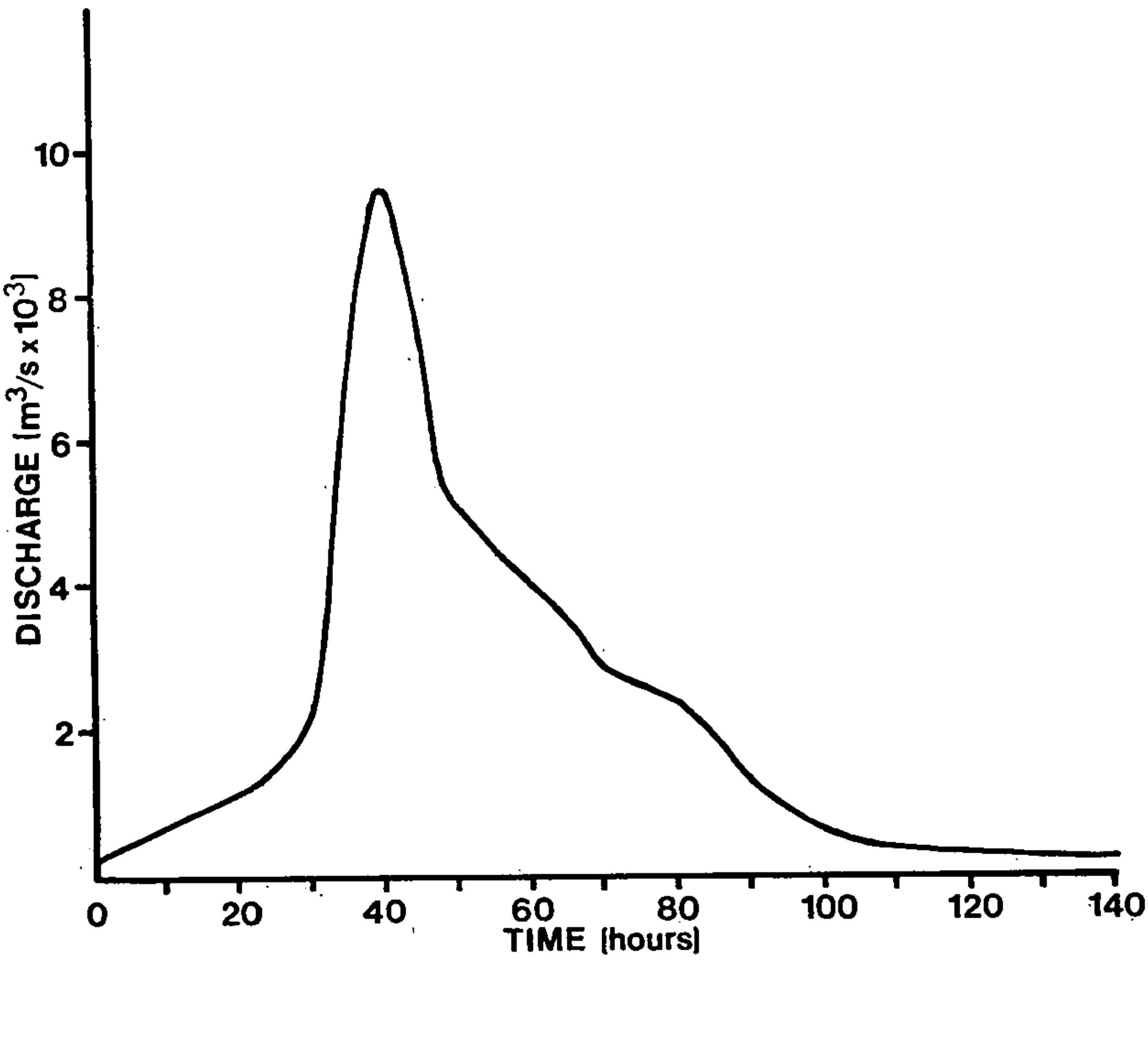


Figure 12

PROBABLE MAXIMUM FLOOD AT ULHITIYA

APPLICATION OF PROBABLE MAXIMUM PRECIPITATION TO UNIT HYDROG

ULHITIA PROBABLE MAXIMUM FLOOD

PERCENTAGE RUNOFF INCREASING THROUGH STORM WITH CWI

AREA (SQ.KM.)	290.0
DATA INTERVAL (HR)	1.0
DESIGN DURATION (HR)	24.0
TOTAL RAIN (MM)	584.4
PERCENTAGE RUNOFF	60.0
ANSF (CUMECS PER SQ.KM.)	.0277
CWI AT START OF STORM	200.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH
.00	15.00	4.04	.00	8.05
1.00	15.00	4.29	4.74	13.60
2.00	15.00	4.54	9.48	25.05
3.00	15.00	4.78	14.22	42.74
4.00	15.00	5.02	18.97	67.02
5.00	15.00	5.24	23.71	98.19
6.00	15.75	5.75	28.45	136.57
7.00	15.70	5.97	33.19	182.86
8.00	15.75	6.23	37.93	237.36
9.00	17.78	7.33	42.67	300.43
10.00	17.78	7.63	47.41	373.57
11.00	40.51	19.26	52.16	457.22
12.00	175.51	122.77	53.75	563.65
13.00	40.51	29.90	50.63	829.41
14.00	17.78	13.25	47.51	1126.26
15.00	15.75	11.81	44.39	1430.79
16.00	15.75	11.88	41.27	1740.74
17.00	15.75	11.95	38.15	2055.11
18.00	15.00	11.42	35.03	2373.44
19.00	15.00	11.47	31.91	2694.16
20.00	15.00	11.51	28.79	3016.79
21.00	15.00	11.55	25.67	3340.04
22.00	15.00	11.60	22.56	3662.23
23.00	15.00	11.64	19.44	3972.65
24.00			16.32	4160.35
25.00			13.20	4152.99
26.00			10.08	4092.68
27.00			6.96	4003.48
28.00			3.84	3887.26
29.00			.72	3743.91
30.00			.00	3576.61
31.00				3387.07
32.00				3175.40
33.00				2941.76
34.00				2686.54
35.00				2409.24
36.00				2121.10
37.00				1838.32
38.00				1561.11
39.00				1290.31
40.00				1026.35
41.00				777.39
42.00				617.90
43.00				504.85
44.00				407.26
45.00				320.67
46.00				244.78
47.00				179.68
48.00				125.03
49.00				80.74
50.00				46.86
51.00				23.43
52.00				10.48
53.00				8.05

CURVATURE AROUND PEAK = -195.046

PROBABLE MAXIMUM FLOOD AT ULHITIYA

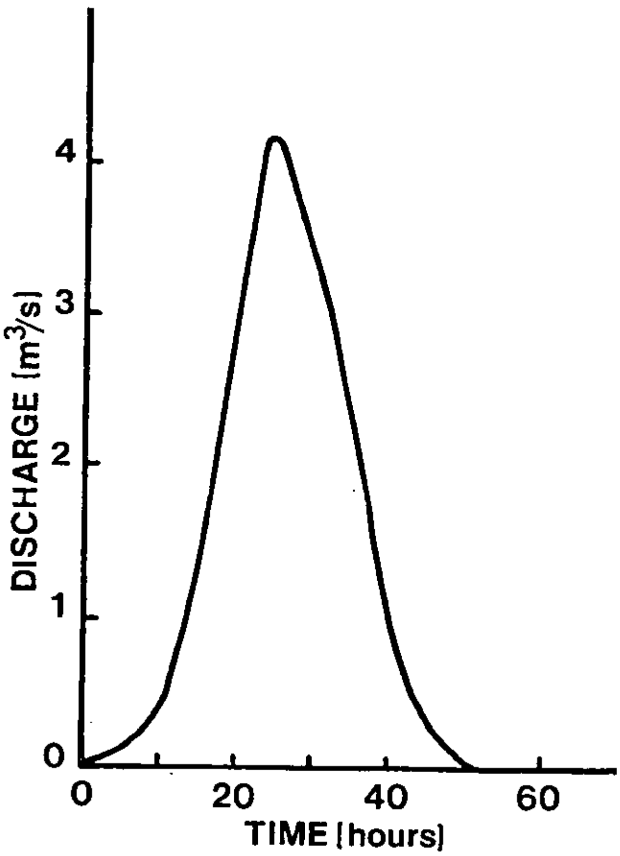


Figure 13

The 1000 year flood at Victoria dam site was also calculated from the 1000 year 2-day rainfall whose derivation is discussed above. The resulting hydrograph is given in Table 12 and gives a peak discharge of 6431 m^3/s. This is comparable with the peak of 5947 m^3/s recorded at Gurudeniya for the storm of August 1947.

These estimates of the PMF for the Victoria and Ulhitiya dams can be used for feasibility design. The analysis at Ulhitiya was made using a number of simplifying assumptions, concerning both the unit hydrograph and the rainfall estimates; the results must therefore be used with full consideration of the limitations inherent in the methods used.

Although the flood estimates for Victoria are based on more rigorous analysis, it must be emphasised that the derivation of the unit hydrograph relied on the rating curve at Gurudeniya whose validity at high flows may be doubtful. Nevertheless, we believe that the results quoted in this section are the best than can be made from the available data.

Mean annual flood

The mean annual flood, $\bar{Q}$, at a site is defined as the arithmetic mean of the recorded series of annual maximum instantaneous discharges. Annual maximum series were available at ten gauging stations within the study area and the calculated $\bar{Q}$ at each of these sites is given in Table 13.

Although there is an apparent discrepancy between the $\bar{Q}$ calculated for Randenigala and the upstream gauges at Peradeniya and Gurudeniya, this can be partly explained by the fact that the shorter period of record at Randenigala only starts in 1955. The record therefore does not include the maximum flood ever recorded at both Peradeniya and Gurudeniya that occurred in 1947. Nevertheless, it is possible that the floods at Randenigala are underestimated but on balance, as it seemed preferable to use all the available records, the Randenigala flood records have been accepted.

1000 YEAR FLOOD AT VICTORIA

APPLICATION OF 1000 YEAR RAINFALL TO UNIT HYDROGRAPHS.

VICTORIA: 1947 FLOOD PROFILE ON AN ESTIMATED 2-DAY RAINFALL.

PERCENTAGE RUNOFF INCREASING THROUGH STORM WITH CWI

AREA (SQ.KM.)	1891.0
DATA INTERVAL (HR)	1.0
DESIGN DURATION (HR)	75.0
TOTAL RAIN (MM)	519.4
PERCENTAGE RUNOFF	60.0
ANSF (CUMECS PER SQ.KM.)	.1370
CWI AT START OF STORM	200.0

CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE

TIME	TOTAL RAIN MM	NET RAIN MM	UNIT HYDROGRAPH ORDINATE	TOTAL HYDROGRAPH
.00	14.40	4.87	.01	259.27
1.00	18.77	6.87	.02	259.46
2.00	17.53	6.86	.07	260.18
3.00	9.38	3.77	.16	251.89
4.00	4.35	1.76	.43	256.35
5.00	3.16	1.27	.63	273.26
6.00	3.16	1.27	1.19	265.33
7.00	1.98	.79	1.89	304.05
8.00	.00	.00	4.61	348.14
9.00	.59	.23	8.12	430.56
10.00	.59	.23	12.89	553.56
11.00	1.98	.76	15.85	722.32
12.00	2.57	.99	18.18	891.58
13.00	4.74	1.84	18.87	1011.59
14.00	6.81	2.68	19.08	1106.16
15.00	8.79	3.55	17.98	1159.31
16.00	8.99	3.71	16.15	1159.45
17.00	10.96	4.67	13.95	1139.41
18.00	6.81	2.93	11.73	1079.16
19.00	1.19	.51	9.87	1008.41
20.00	32.49	15.43	8.27	941.35
21.00	23.11	11.69	7.21	893.98
22.00	24.49	13.18	6.35	873.96
23.00	25.08	14.32	5.79	890.37
24.00	25.08	15.11	5.34	945.08
25.00	62.21	43.24	5.01	1032.66
26.00	33.87	24.99	4.72	1147.54
27.00	8.79	6.48	4.50	1276.23
28.00	20.64	15.62	4.29	1456.80
29.00	24.89	19.49	4.04	1697.37
30.00	16.00	12.70	3.70	2025.58
31.00	7.70	6.08	3.34	2408.97
32.00	14.32	11.42	3.03	2659.15
33.00	3.46	2.72	2.81	3428.23

TABLE 12

(continued....)

34.00	2.57	1.99	2.63	4093.98
35.00	3.65	2.79	2.46	4793.26
36.00	1.48	1.11	2.36	5377.48
37.00	1.48	1.09	2.26	5839.42
38.00	4.94	3.61	2.17	6176.64
39.00	4.94	3.58	2.08	6383.27
40.00	2.57	1.84	1.97	6431.25
41.00	5.14	3.64	1.88	6335.80
42.00	3.16	2.21	1.79	6065.50
43.00	8.99	6.30	1.71	5725.17
44.00	6.22	4.34	1.61	5338.92
45.00	6.42	4.46	1.51	4921.95
46.00	4.74	3.26	1.40	4529.40
47.00	5.53	3.78	1.32	4174.99
48.00	2.96	2.00	1.25	3874.90
49.00	2.37	1.56	1.18	3636.59
50.00	3.36	2.21	1.11	3445.57
51.00			1.05	3316.14
52.00			.98	3227.03
53.00			.89	3164.29
54.00			.80	3143.96
55.00			.71	3104.91
56.00			.61	3046.27
57.00			.53	2977.72
58.00			.45	2887.64
59.00			.40	2774.62
60.00			.35	2639.13
61.00			.30	2483.94
62.00			.24	2322.93
63.00			.18	2160.26
64.00			.12	2006.77
65.00			.05	1862.16
66.00			.02	1733.17
67.00			.00	1619.14
68.00				1519.97
69.00				1431.67
70.00				1352.62
71.00				1282.19
72.00				1217.90
73.00				1157.78
74.00				1099.79
75.00				1044.03
76.00				989.69
77.00				937.71
78.00				886.41
79.00				837.06
80.00				789.98
81.00				745.23
82.00				702.95
83.00				662.62
84.00				625.38
85.00				589.66
86.00				555.57
87.00				523.13
88.00				492.78
89.00				464.78
90.00				438.85

TABLE 12
(continued....)

91.00	416.35
92.00	397.12
93.00	380.69
94.00	366.01
95.00	353.10
96.00	342.08
97.00	332.54
98.00	324.27
99.00	316.73
100.00	309.88
101.00	303.33
102.00	297.35
103.00	291.70
104.00	286.56
105.00	281.68
106.00	277.66
107.00	273.68
108.00	270.42
109.00	267.50
110.00	265.04
111.00	263.13
112.00	261.64
113.00	260.58
114.00	259.67
115.00	259.44
116.00	259.23
117.00	259.16

CURVATURE AROUND PEAK = -173.418

TABLE 13

RECORDED MEAN ANNUAL FLOODS

River	Station	Area (km²)	Mean catchment rainfall (mm)	Mean annual flood (m³/s)
Mahaweli Ganga	Peradeniya	1189	3118	1259
" "	Gurudeniya	1417	2934	1413
" "	Randenigala	2370	2713	913
" "	Manampitiya	7340	2504	3084
Hulu Ganga	Teldeniya	161	3273	252
Galmal Oya	Moragamulla	73	2731	147
Uma Oya	Talawakanda	505	2064	295
Maha Oya	Hanguranketa	105	2553	227
Gallodai Aru		223	2144	133
Maduru Oya		453	2173	355

At ungauged sites, $\bar{Q}$ can be estimated from a relationship between the observed flood series and catchment characteristics. For this analysis we have used the ten stations given in Table 13 to derive the form of the regional relationship using correlation analysis between $\bar{Q}$, and catchment area and rainfall.

The results of the regression analysis are given in Table 14, and lead to the following prediction equation:

$$\bar{Q} = 3.846.\ A^{0.668}.\ R^{1.837} \times 10^{-6}$$

where $\bar{Q}$ is the mean annual flood in m^3/s
A is the catchment area in km^2
and R is the mean annual catchment rainfall in mm.

The standard error of the equation is 0.129 in logarithmic form or +34 per cent, -26 per cent in multiplicative form. The mean annual flood, $\bar{Q}$ is calculated as 1304 m^3/s and 224 m^3/s for the Victoria and Ulhitiya dam sites respectively. At Gurudeniya, the observed flood is 1413 m^3/s, and the predicted flood 1148 m^3/s, giving a ratio of observed to predicted of 1.23. In order to maintain consistency with the observed $\bar{Q}$ at Gurudeniya we consider that the predicted $\bar{Q}$ at Victoria should be adjusted by this ratio. The estimated mean annual flood at Victoria is therefore 1604 m^3/s.

Regional flood frequency curve

When a flood, Q(T), of moderate return period T years is required for design purposes, it may be determined either by statistical analysis of a sufficiently long series of recorded floods at the site or it may be deduced from the mean annual flood, $\bar{Q}$ and the regional flood frequency curve. This is generally a composite curve in which the ratio $Q(T)/\bar{Q}$ is plotted against the return period T; the use of this type of curve is described in detail in the FSR.

Briefly, once the regional curve has been drawn, and the $\bar{Q}$ calculated from the data or estimated using the prediction equation,

TABLE 14

REGRESSION OF $\bar{Q}$ ON AREA (A) AND MEAN ANNUAL RAINFALL

No. of variables	Name	Coeff	seb			r^2	see	Const.	Mult.
		0.677	0.094	7.2	0.930	0.865	0.181	0.832	6.792
	A	0.668	0.067	9.9	0.970	0.940	0.129	-5.415	3.846×10^{-6}
	R	1.837	0.622	3.0					

Notes:

- Coeff is the regression coefficient in an equation of the form $\bar{Q}$ = constant $A^{b1}.R^{b2}$,
- seb is the standard error of estimate of b,
- t is Student's t statistic,
- r and r^2 are the coefficients of multiple correlation and determination,
- see is the standard error of estimate of the relationship; the factorial standard error of estimate is the antilogarithm of see,
- Const. is the intercept of the regression equation
- Mult. is the antilogarithm of Const. for use in the multiplicative form.

Q(T) can be inferred. And although it is obviously essential to use this approach at sites with short or no records, it can also be useful at sites where the records are long enough for conventional statistical analysis.

Experience in different parts of the world has shown that the variations between different sample periods of a single record are similar to the variations between the records at different stations within a region. A composite regional flood frequency curve therefore gives a more reliable relationship between Q(T) and $\bar{Q}$ than the simple extrapolation of the flood frequency curve at a station with a short record.

A regional curve has been constructed from the ten series of annual maximum floods at the sites in or near the project area that are given in Table 15 together with the corresponding $\bar{Q}$. It is important to note that in several cases, the series included an outlier which will affect the calculated mean. Each record was converted into a dimensionless series $Q/\bar{Q}$ and the individual events ranked in ascending order. The plotting position, y_i, that corresponds to the flood of rank i in the series, was esimated from the Gringorten formula, which is given by:

$$F_i = \frac{i - 0.44}{N + 0.12}$$

and

$$y_i = -\ln -\ln F_i$$

where F_i is the plotting position expressed as a probability,
i is the rank of the event,
and N is the number of events in the series.

These floods were then grouped into ranges of y(-1.5 to -1.0 etc), and the mean values of y and the ratio $Q/\bar{Q}$ calculated for each range. By using these calculated means, it was possible to define the regional curve up to a value of y = 4.0, or a return period of about 50 years. The curve may then be tentatively extended further by plotting the four highest individual values of $Q/\bar{Q}$ as being the four

STATIONS USED TO DERIVE THE REGIONAL FLOOD FREQUENCY CURVE

River	Site	Years of data	Mean annual flood, $\bar{Q}$ (m^3/s)	$Q\ max/\bar{Q}$
Mahaweli Ganga	Peradeniya	33	1259	4.05
"	Gurudeniya	33	1413	4.21
"	Randenigala	23	913	1.80
"	Manampitiya	24	3084	6.11
Hulu Ganga	Teldeniya	23	252	3.15
Galmal Oya	Moragamulla	14	147	1.73
Uma Oya	Talawakanda	19	295	3.99
Maha Oya	Hanguranketa	5	227	1.87
Gallodai Aru		27	133	3.82
Maduru Oya	Kandengama	6	355	2.37

Note: Qmax is the peak discharge recorded at each station.

highest events taken from a sample population of 207.

A general extreme value curve given by

$$Q/\bar{Q} = u + \alpha\left(\frac{1-e^{-ky}}{k}\right)$$

with values of

$u = 0.70$
$\alpha = 0.30$
and $k = -0.32$

fits these points well. This curve, shown in Figure 14, can be used to deduce $Q/\bar{Q}$ for rivers in the region for return periods of up to about 50 years.

Construction floods

For construction purposes, it is necessary to estimate the probability of exceedance of a flood of given magnitude during an individual calendar month or season. Ideally, this could be deduced from analysis of monthly and seasonal instantaneous discharges, Q. These data were not immediately accessible from the Irrigation Department files, but as the monthly maximum average daily flows, q, are tabulated for each gauging station, the latter have been used instead. The two long-term gauging stations at Gurudeniya and on the Gallodai Aru were chosen as the most suitable stations for construction flood analysis at the Victoria and Ulhitiya dam sites respectively.

The frequency distributions of the series of annual maximum average daily flows at each gauging station were first compared with the distribution defined by the regional flood frequency curve. Each series was ranked, and the magnitude of each event in the series expressed as a fraction of its mean. The individual points were then plotted and the resulting frequency distribution compared with the regional curve (Figure 14). As there was good agreement between the curves, we have assumed that the distribution of the monthly and seasonal average daily discharges will also be similar to the corresponding series of instantaneous discharges.

REGIONAL FLOOD FREQUENCY CURVE

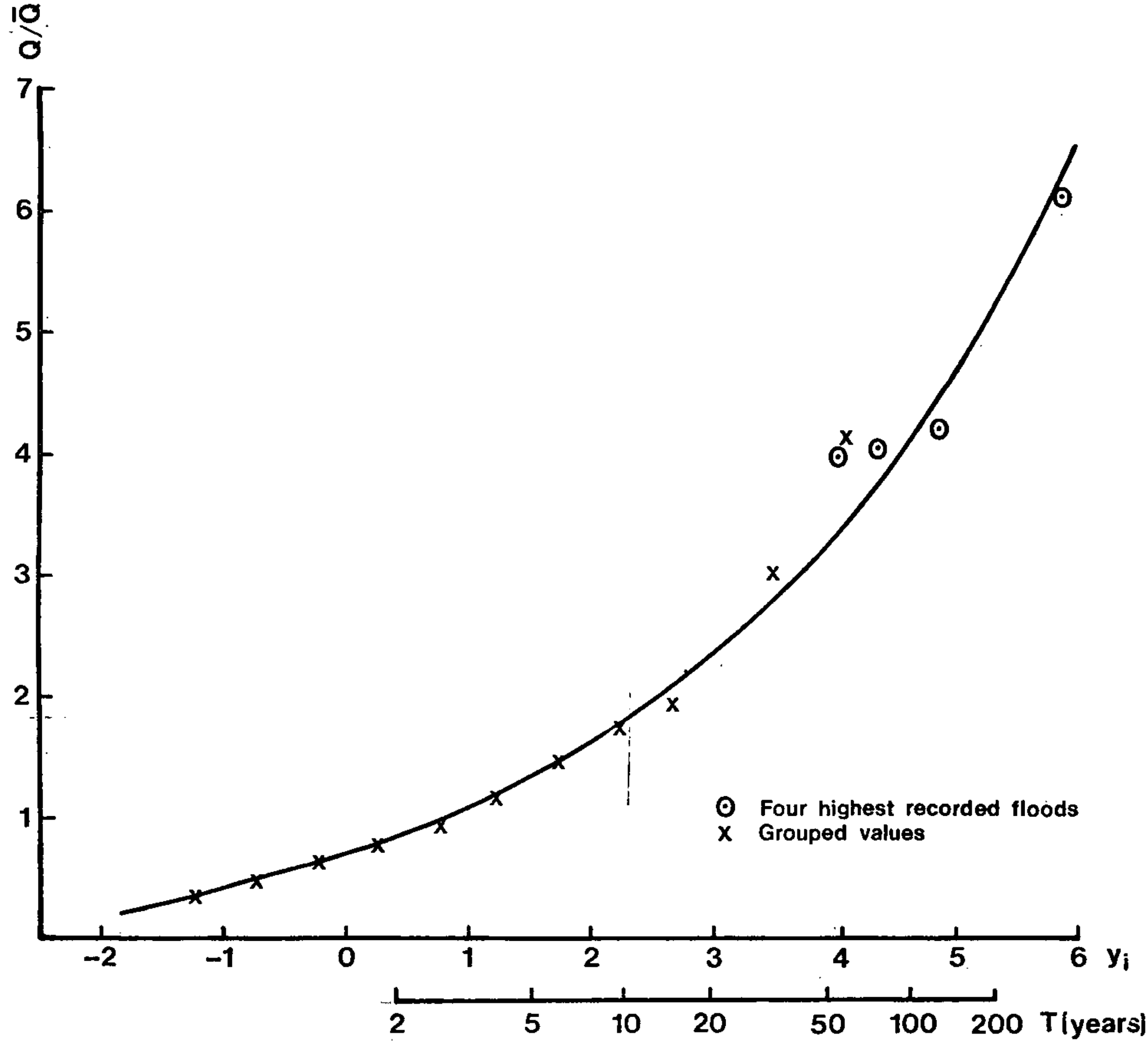


Figure 14

In Table 16 the means of the series of monthly maximum average daily flows, q, are given for Gurudeniya and the Gallodai Aru, together with the maximum average daily flow recorded in each month. These records suggest that there is a four month period from January to April when the mean, and maximum, floods at Gurudeniya were considerably lower than during the rest of the year. A similar dry period on the Gallodai Aru occurs from May to September.

It follows that for each river the probability of a flood of given magnitude being exceeded during the dry season will be lower than during the rest of the year. Consequently the monthly and seasonal exceedance probabilities were estimated separately for the dry season on each river. Thus for each month of its dry season, the q at Gurudeniya were ranked and plotted. A Gumbel Type 1 extreme value distribution was then fitted to the points by eye as a straight line. The probability of non-exceedance for floods whose magnitude is a given fraction of the mean annual maximum average daily flow, $\bar{q}$, was then determined for each month. The seasonal probability of non-exceedance may then be estimated as the product of the monthly probabilities. The monthly and seasonal probabilities of exceedance can then be deduced; these are expressed in terms of return period in Table 17 and are shown graphically in Figure 15.

On the Gallodai Aru, zero flows have been recorded for a number of months during its dry season. Consequently, a graphical analysis of the monthly maximum average daily flows was not possible. The series of maximum flows from the whole dry season was used instead, and following the analysis described above, gives the results shown in Table 18 and Figure 16.

During the dry season, the magnitude of a flood of given return period at each site can then be found from the appropriate graph (Figure 15 for Victoria and Figure 16 for Ulhitiya) in terms of the mean annual flood $\bar{Q}$ that has been calculated earlier. During the remainder of the year, May to December at Victoria and October to April at Ulhitiya, we have assumed that the annual flood has an equal probability of occurring at any time; the regional flood frequency curve (Figure 14) should then be used.

MONTHLY MAXIMUM AVERAGE DAILY FLOWS
(m^3/s)

GURUDENIYA

	O	N	D			M		M				S	Annual
Mean	402	296	333	120	74	76	118	261	395	371	388	321	859
Maximum recorded	1066	792	2921	427	224	308	262	1000	1817	1932	3427	1124	2921

GALLODAI ARU

Mean	11	30	69	62	42	12	11						86
Maximum recorded	82	62	309	96	260	64	54	17	17	28	17	12	309

TABLE 17

ANALYSIS OF MONTHLY AND SEASONAL FLOODS AT GURUDENIYA

	Jan	Feb	Mar	Apr	Season (Jan-Apr)
$0.25 \times \bar{q}$					
Prob. of non-exceedance	0.8476	0.9730	0.9581	0.9318	0.7363
Prob. of exceedance	0.1524	0.0270	0.0419	0.0.682	0.2637
Return period (years)	6.6	37.0	23.9	14.7	3.79
$0.30 \times \bar{q}$					
Prob. of non exceedance	0.9090	0.9884	0.9794	0.9703	0.8538
Prob. of exceedance	0.0910	0.0116	0.0206	0.0297	0.1462
Return period (years)	11.0	86.2	48.5	33.7	6.84
$0.35 \times \bar{q}$					
Prob. of non exceedance	0.9438	0.9951	0.9903	0.9872	0.9182
Prob. of exceedance	0.0562	0.0049	0.0097	0.0128	0.0818
Return period (years)	17.8	204	103	78.1	12.2
$0.40 \times \bar{q}$					
Prob. of non exceedance	0.9655	0.9980	0.9953	0.9948	0.9541
Prob. of exceedance	0.0345	0.0020	0.0047	0.0052	0.0459
Return period (years)	29.0	500	213	192	21.8

Note: $\bar{q}$ is defined as the mean annual maximum average daily flow

FLOOD FREQUENCY CURVE FOR CONSTRUCTION FLOODS AT VICTORIA

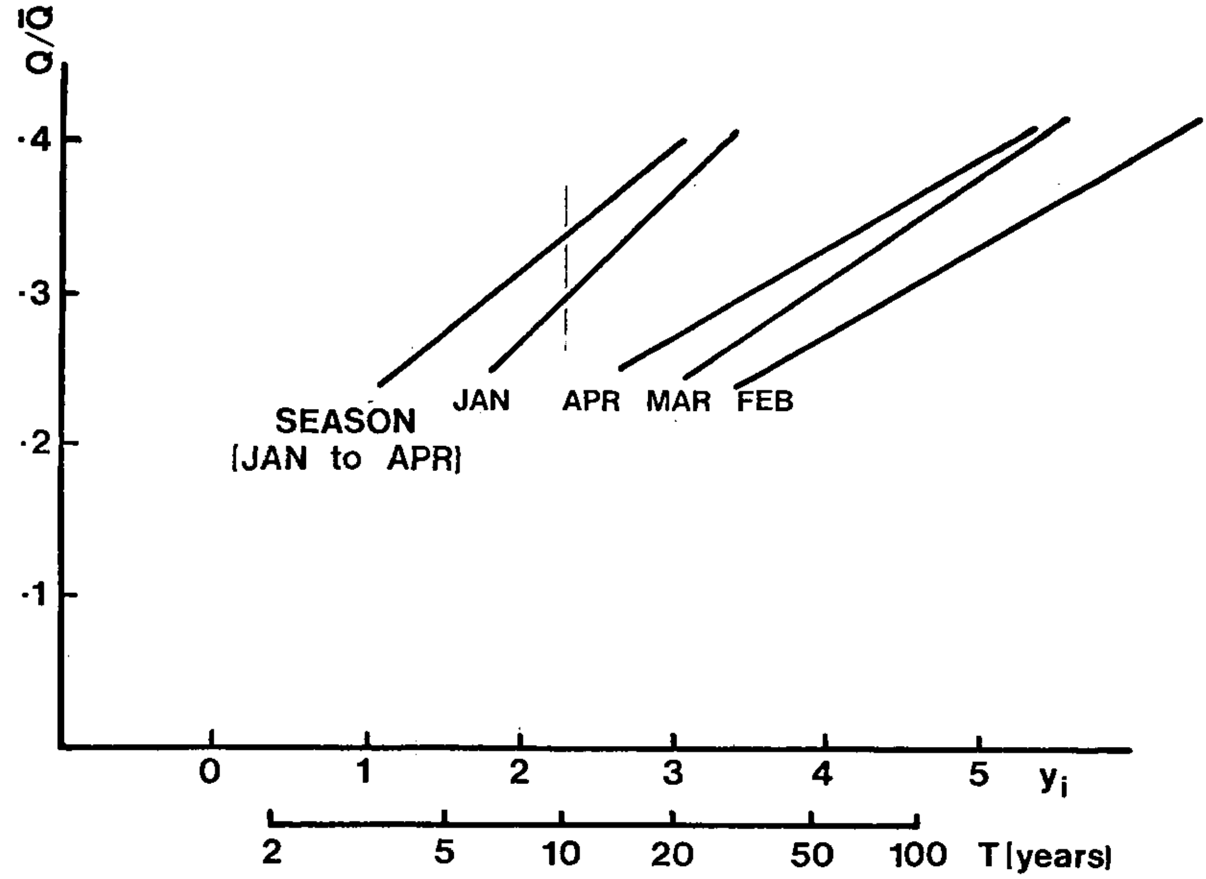


Figure 15

TABLE 18

ANALYSIS OF SEASONAL FLOODS ON THE

GALLODAI ARU

Fraction of $\bar{q}$	Probability of exceedance	Return period (years)
0.10	0.3341	3.0
0.15	0.1524	6.6
0.20	0.0650	15
0.25	0.0270	37

Note: $\bar{q}$ is defined as the mean annual average daily flow

FLOOD FREQUENCY CURVE FOR CONSTRUCTION FLOODS AT ULHITIYA

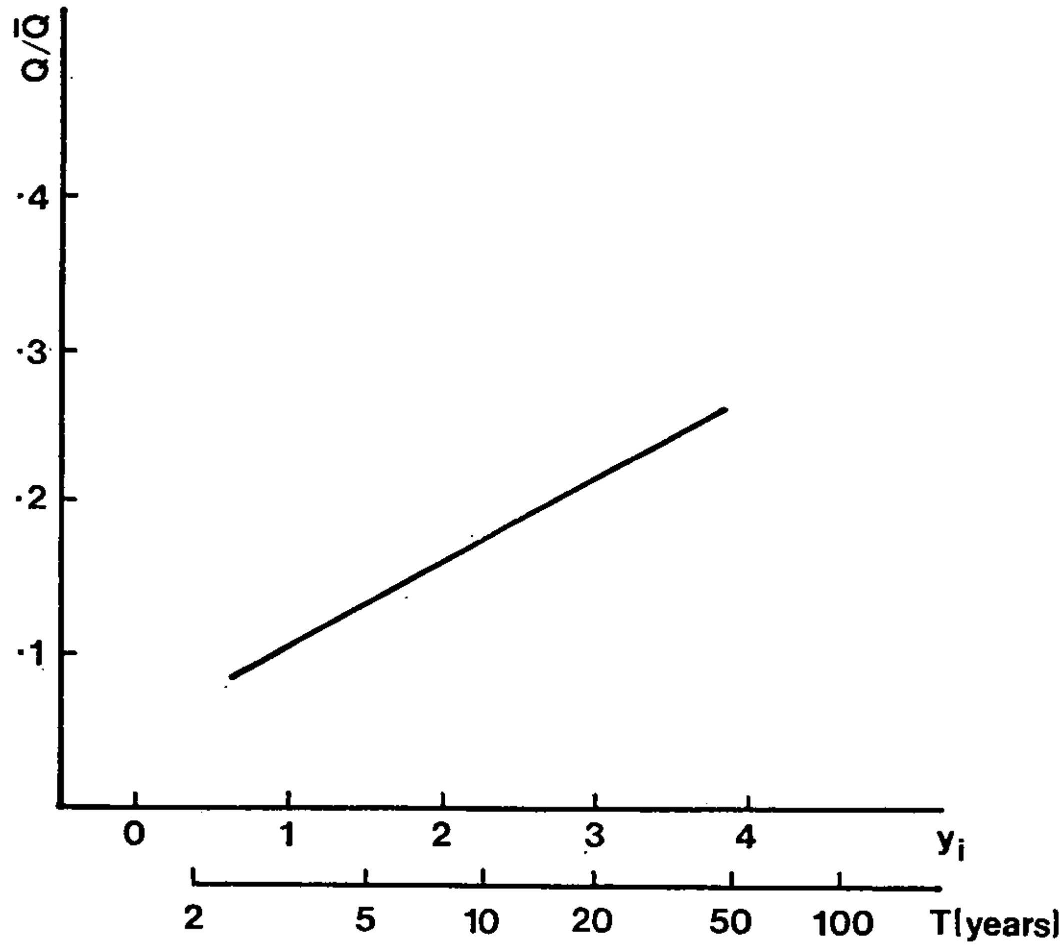


Figure 16

A simplified triangular unit hydrograph can be used to deduce the hydrograph or shape of a given construction flood at Victoria. From estimates of the catchment rainfall and storm runoff for a short return period flood, a unit hydrograph was deduced with the following characteristics:

T_p, time to peak 14 hours
T_B, time of base 33 hours.

Using a value of $\bar{Q}$ = 1605 m^3/s we can estimate the 20 year and 10 year annual floods at Victoria as 3690 m/s and 2900 m^3/s respectively. From T_p and T_B the corresponding hydrographs can be found as illustrated in Figure 17.

This analysis is intended to provide a guide from which the risks of flooding during construction at either site can be assessed; the results of the analysis should therefore be used with caution and even then only up to return periods of 25 years.

SIMPLIFIED UNIT HYDROGRAPHS FOR CONSTRUCTION FLOODS AT VICTORIA

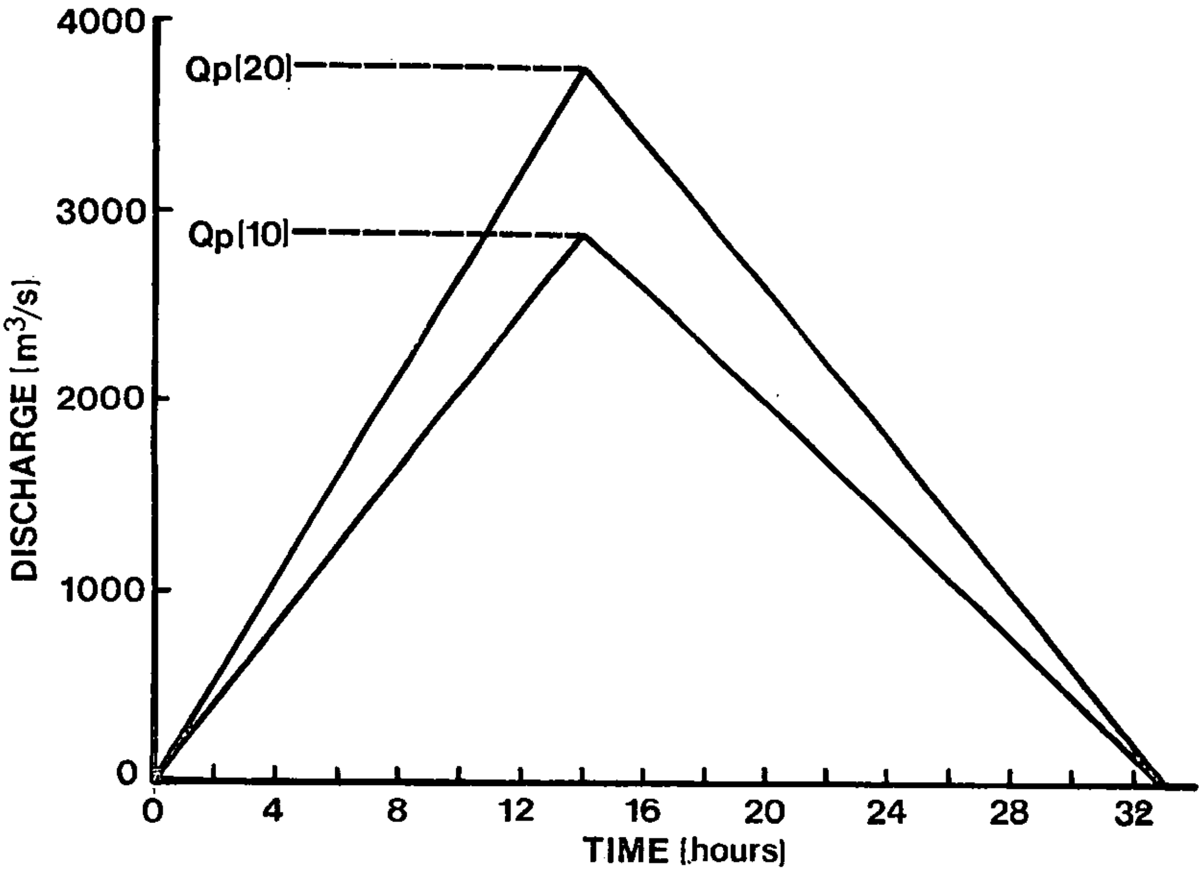


Figure 17

SUMMARY AND RECOMMENDATIONS

We consider that the monthly flow data given in Tables 6 and 7, and the spillway design floods given in Tables 10 and 11, should be used as the hydrological data for preliminary engineering design. The information in these tables is the best that could be derived from the data in the limited time available for the study. There are however a number of limitations in the quality and accuracy of the basic data which could be overcome easily. We include some recommendations, which apply mainly to the measurement of extreme events, and which would help achieve this aim.

The importance of verifying the extrapolation of the rating curves of all the gauging stations cannot be over-emphasised, because the reliable estimation of high flows is vital for both reservoir yield and operational analysis as well as flood design. It is clearly impossible to gauge floods safely or accurately on the Mahaweli Ganga without cableway and suspended current meter installations. But it is possible to estimate peak discharges by using a relationship such as Manning's equation. Such an approach involves a survey of the river cross-section and measuring the slope of the water surface either from points defined during the flood or later from flood debris. This information is used with an estimate of roughness coefficient, determined from previous gaugings at lower flows or by analogy with published data, to calculate the surface velocity of the peak flow; it may also be possible to measure the velocity during the flood with simple floats. This type of investigation is relatively simple and would enable the extrapolated rating curves to be verified.

It is clear that any major changes in the rating curve for high flows at Gurudeniya might modify the ordinates of the derived unit hydrograph and hence alter the magnitude of the spillway design and 1000 year floods given in Tables 10 and 12. These results rely on a simplified approach to the estimation of the Probable Maximum Precipitation using moisture maximization. A more detailed meteorological analysis would enable this work to be verified before the final engineering designs are made. We would however expect any revisions to Tables

10 and 12 resulting from more detailed meteorological analysis to be minor.

The estimated monthly flows on the Ulhitiya Oya and the right-bank tributaries are based on extremely limited data and rely on several very general assumptions. The yield and flow characteristics of these rivers should be verified as soon as possible by stage measurements and gaugings.

Moreover the spillway design flood at Ulhitiya (Table 11) is based on further assumptions. Although these have been made from all the available local information, the design flood had nevertheless to be based on empirical relationships. In particular, the estimates of catchment lag, as well as the design storm depth and duration, should be verified. We therefore recommend that either a recording gauge should be installed, or that hourly gauge readings should be re-started, at a suitable point on the Ulhitiya Oya. Such a programme of data collection is vitally important and the costs incurred would be very small in comparison to the total costs of the scheme.

Although there is a sparse network of raingauges and no climatological stations in this north eastern part of the study area, improvements in the collection of rainfall and evaporation data are not so critical in the short term. In view of the extensive areas to be developed for irrigation, it would be desirable to consider the installation of a climatological station in this region at an early date. During the period between the presentation of this report and the dam construction, particular emphasis should be placed on the measurement of intense, short-duration rainfall. Such information would enable the design storm, and hence the PMF, to be determined more precisely.

ANNEX

The model chosen for extending the data is a multivariate lag-one Markov model. This type of model enables cross-correlations between concurrent flows at a number of gauging stations (ie, lag-zero cross correlations), as well as lag-one serial correlations within the individual station records, to be preserved in the extended flow sequences. For n sites the model equation takes the general matrix form:

$$\underline{X}_t = \underline{A}\,\underline{X}_{t-1} + \underline{B}\,\underline{\varepsilon}_t,$$

where $\underline{X}_t$ is an (n x 1) vector of standardised values at time t,
$\underline{A}$ and $\underline{B}$ are (n x n) matrices containing the parameters of the model,
and $\underline{\varepsilon}_t$ is an (n x 1) vector of independent random elements.

Assuming that the matrix $\underline{A}$ is diagonal and that the matrix $\underline{B}$ has a lower triangular form, the model equation for three sites can be written in full as:

$$x_t = a_{11}\,x_{t-1} + b_{11}\varepsilon_t^x$$

$$y_t = a_{22}\,y_{t-1} + b_{21}\varepsilon_t^x + b_{22}\varepsilon_t^y$$

$$z_t = a_{33}\,z_{t-1} + b_{31}\varepsilon_t^x + b_{32}\varepsilon_t^y + b_{33}\varepsilon_t^z$$

where x_t, y_t and z_t are the flows at each of the three sites at time t,
and x_t is the long-term record.

The lag-one serial correlation for each flow series is preserved by the elements of the $\underline{A}$ matrix and the lag-zero cross correlations between the series are maintained through the elements of the $\underline{B}$ matrix.

As this type of analysis can only be applied to what are known as stationary series, seasonal effects have to be removed. Before this can be done, a logarithmic transformation is applied to the basic flow data, q_t, to give a normalised series Q_t, where Q_t is given by:

$$Q_t = \log_e q_t$$

The transformed flows in month i, ie the Q_t^i, are then standardised by removing the mean (μ_i) and standard deviation (σ_i) to give X_t^i as follows:

$$X_t^i = \frac{(Q_t^i - \mu_i)}{\sigma_i}$$

The lag-zero cross correlations and lag-one serial correlations were calculated for each month of the common period between the stations. The corresponding correlation coefficients were also calculated for the lumped series of monthly flows at each station. It was found that, with the exception of one value, the monthly coefficients were not significantly different from the coefficients of the lumped series at the 95 per cent confidence limits. The exception was the serial correlation coefficient at Talawakanda for the period September to October. We therefore decided to split the year into two periods to take account of this fact.

The matrices $\underline{A}$ and $\underline{B}$ were then calculated separately for the periods November to August, and September to October. The $\underline{A}$ matrix contains the lag-one serial correlation coefficients as its diagonal terms, and thus preserves the serial correlation in the generated series.

The calculation of the elements of the $\underline{B}$ matrix is more involved, and relies on the equation:

$$\underline{B}\,\underline{B}^T = \underline{Mo} - \underline{A}\,\underline{Mo}^T\,\underline{A}$$

where $\underline{Mo}$ is the lag-zero cross correlation matrix,

and the superscript T denotes the transpose of the matrix.

From knowledge of the elements of the ($\underline{B}\,\underline{B}^T$) matrix, the elements of $\underline{B}$, which is assumed to be lower triangular, can then be calculated.

When generating the extended flow records, the equations quoted above can be rearranged, and become:

$$\varepsilon_t^x = (x_t - a_{11}x_{t-1})/b_{11}$$

$$y_t = a_{22}y_{t-1} + b_{21}\varepsilon_t^x + b_{22}\varepsilon_t^y$$

$$z_t = a_{33}z_{t-1} + b_{31}\varepsilon_t^x + b_{32}\varepsilon_t^y + b_{33}\varepsilon_t^z$$

thus making use of the long-term record, x_t.

The elements ε_t^y and ε_t^z were generated as pseudo random numbers, taken from a normal distribution with zero mean and unit variance, to account for the unexplained variance in the extended flows. Once the y_t and z_t had been extended to cover the period of the x_t, the individual values were then transformed to have the historic mean and variance of the recorded data.

REFERENCES

1. Report on the Colombo observatory for 1967 (with maps and statistics) L.A.D.I. Ekanayake 1972.

2. Baghirathan, V.R., and Shaw, E.M.; Rainfall - Depth - Duration - Frequency Studies for Sri Lanka. J. Hydrol., 1978, 37(3/4), 223-239.

3. Annual rainfall and surface runoff; Irrigation Department, Sri Lanka, 1974.

4. Hydrological Annual 1973/74; Irrigation Department, Sri Lanka, 1976.

5. A system for the quality control and processing of streamflow, rainfall and evaporation data; Institute of Hydrology, Report No. 15, 1974.

6. Bowatenna Diversion Unit; SOGREAH 1971.

7. The behaviour of the Yala and Maha Season rainfall in Sri Lanka during the last eighty years (1896-1975); Harihara Ayyar, P.S., Meteorological Department.

8. Development of design flood hydrographs for ungauged catchments using Snyder's technique, as adopted to catchments of Sri Lanka; Mahaweli Development Board, 1976.

9. Flood Studies Report; Natural Environment Research Council, 1975.

10. Hershfield, D.M.; Estimating the probable maximum precipitation. Proc A.S.C.E., J. Hydraul. Div. 1961, 87, 99-116.

11. Snyder, F.F.; Synthetic Unit Hydrographs. Trans. Am. Geophys. Union, 1938, 19(1), 447-454.

12. An analysis of rainstorms in the Upper Mahaweli Catchment. Unpublished work, P. Sumanasekere, Meteorological Department.

13. Manual for estimation of probable maximum precipitation; Operational Hydrology Report No. 1, WMO Geneva, 1973.

14. Hydrometeorology, C.J. Wiesner, Chapman and Hall, 1970.

REPORT ON A VISIT TO MALAWI

APRIL 7-APRIL 21 1978

by

DR. E.P. WRIGHT

Report No. WD/OS/78/26 July 1978

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REPORT ON A VISIT TO MALAWI APRIL 7-APRIL 21 1978.

INTRODUCTION

1.1 The Government of Malawi has requested a 2/3 month assignment by a consultant hydrogeologist for the purpose of formulating appropriate studies which will enable the groundwater resources of the country to be evaluated. In view of the broad terms of reference for an assignment of quite short duration, a preliminary visit by the writer was deemed advisable to review the circumstances and to identify the scope of the problem.

BACKGROUND TO REQUEST

2.1 Groundwater is currently utilised from drilled boreholes (3500+), numerous shallow dug wells and occasional springs. Wells are drilled or dug in accordance with local requirements and although there are no significant occurrences of supply deficiency due to demand, in certain areas, e.g. the Rift Scarp, suitable borehole sites are difficult to locate. Boreholes and wells are generally scattered and although abstraction has not been quantified, it is unlikely to be in excess of recharge except perhaps locally. Usage is mainly for domestic water supply and it has been estimated by the World Health Organisation (Water Supply and Sewerage Sector Study, Republic of Malawi, 1978) that 70% of urban dwellers and 30% of the rural population has access to protected services. During the next few years, it is intended to extend these facilities and by 1980 it is anticipated that 50% of the rural population will have such access. A particular emphasis at the present time is being placed on piped gravity systems and on a shallow hand dug well programme by self help methods. There is concern in Government over the costs of drilling and maintenance of boreholes and the emphasis on the two alternative methods of supply referred to above reflects this attitude.

2.2 An important recommendation in the 1978 WHO report (op.cit) is for the formulation of a National Water Resources Master Plan. The plan must consider both ground and surface water supplies and will require an adequate knowledge of resource availability and estimates of cost/

benefits of utilisation in order to obtain proper comparisons. There will be some difficulties in obtaining sufficient information on groundwater resources. The supply aquifers are mostly low yielding and heterogeneous and costs of exploration tend to be high relative to the return. Adequate investigations may be difficult to justify in economic terms unless a high demand can be recognised (e.g. industrial, municipal supply, irrigation), in areas where no obvious alternative supply source exists.

2.3 The information required for planning may be provisionally sub-divided into short and long term requirements although there is not wholly a clear distinction between the two. Short term aspects relate mainly to provision of preliminary resource assessments and possible improvements of current development practices, considerations of which might significantly affect immediate planning.

(i) Short term requirements : an analysis of boreholes and dug wells to include considerations of siting, drilling/digging, completion and pumping equipment, water quality and productivity in relation to demand etc. with a view to optimising benefits and reducing costs to a minimum; broad resource assessments using existing hydrological and climatological data combined with well water level records and abstraction estimates.

(ii) Long term requirements : determination of means to make most effective assessment of available groundwater resources consistent with the economics of planned usage. Procedural methods will clearly include analysis of available record data, studies on methodology and detailed resource evaluation of areas where the information is most needed.

2.4 A brief note on the occurrence of groundwater in Malawi follows below. During the planned assignment, it may be feasible to initiate some relevant studies, particularly on the analytical aspects, as well as to formulate plans for the main programme.

ITINERARY

3.1 The writer arrived in Blantyre on April 7th and departed for London on April 21st. Field visits were made to the Central Lilongwe-Kasungu region, the Central Lake Shore area and Bwanje valley, Mlanje and the Lower Shire. Discussions were held with the following people and their assistance is gratefully acknowledged. Particular acknowledgement must be made to Mr. Patrick Marcello of the Geological Survey who made all local arrangements and accompanied the writer throughout the visit.

(i) Ministry of Agriculture and Natural Resources (M.A.N.R.) Headquarters at Lilongwe.

P. Brown, Deputy Secretary
Mr. Sekwese, Senior Project Officer in Planning Section
Mr. Kalua, Principal Administration Officer
A.N. Mandeville, Pricipal Hydrologist of Water Resources Division
Mr. Kindena, Project Manager of the Central Region Lake Shore Project, Salima.

(ii) Geological Survey Department (M.A.N.R.), Zomba.

M.J. Crow, Superintending Geologist
F.W.P. Chapusa, Senior Geologist
F.R.M. Phiri, Senior Geologist
R.R. Muma, Geologist
P.J. Marcello, Geologist
G. Kayira, Geologist
M.Y. Mwenda, Senior Laboratory Assistant
A.G. Mwangande, Drilling Superintendent
W.R. Gibson, Driller
T.E. Chamley, Senior Wells Maintenance Officer (and other staff in the two Borehole Maintenance teams visited at Salima and Ngabu)

(iii) Ministry of Community Development and Social Affairs, Lilongwe.

Mr. Robertson, Senior Engineer

(iv) British High Commission.

Mr. Clapham, First Secretary (Aid)

M. Todd, Engineering Adviser

(v) Shire Valley Agricultural Development Project, Ngabu.

J.H. Stael, Resident Hydrologist

(vi) Private Drilling Contractor.

Drilling Superviser in Kasungu Region

4. MAIN AGENCIES CONCERNED WITH WATER SUPPLY

4.1 Administration

(i) Water Resources Board (MANR). Licences abstractions. Has representatives of various Government Departments, agricultural interests and users.

4.2 Investigation

(i) Water Resources Division (MANR). Executive branch of the WRB; in practice concerned almost exclusively with surface water occurrence and main activity at present concerns maintenance of gauging network.

(ii) Geological Survey Department (MANR). Borehole drilling either with own equipment or by license to private contractor.

(iii) Water Project Section of the Ministry of Community Development and Social Welfare. Gravity piped water supplies and shallow wells using self help methods.

4.3 Development, Operation and Maintenance

(i) Various local Water Boards (Blantyre and Lilongwe).

(ii) Ministry of Works and Supplies (MWS)

(iii) District Councils

(iv) Geological Survey Department (MANR). Borehole maintenance

5. MALAWI : GENERAL

5.1 Malawi is long and narrow with a total land area of 94,000 km^2 and a population of over 5 million. Agriculture is the main economy occupying some 85% of the labour force and contributing some 50% of the GNP. The climate is tropical continental with a dry season from May to November. The greater part of Malawi (c. 60%) has a rainfall between 750 and 1000 mm, 35% in excess of 1000 mm and 5% less than 750 mm.

5.2 There are three distinct physiographic sub-divisions which include (i) the Rift Valley and Lake Malawi (elevation less than 500 metres above mean sea level), (ii) the High Plateaux (1200 to exceeding 1500 metres above mean sea level) and (iii) the dissected Rift Scarp.

5.3 Malawi is largely underlain by crystalline metamorphic and igneous rocks with other consolidated rocks limited to minor occurrences of Karroo sediments and volcanics. There are variable thicknesses of Post-Karroo alluvial deposits occurring mainly within the Rift Valley. Surface colluvium and weathering materials are thickest on the broad upland Plateaux and thin to absent on the eroded Rift Scarp region.

5.4 Surface water occurrences include Lakes Malawi, Malombe and Chilwa. Perennial rivers include the Shire and certain rivers draining from the highland areas of Mlanje and Nyika. Elsewhere surface flow is mostly ephemeral.

6. GROUNDWATER OCCURRENCE

6.1 Rainfall in Malawi is fair to good and maintains a continuous vegetative cover and recharges the shallow aquifers (300 feet) within the weathered bedrock/colluvium and the rift valley alluvial sequences. There are no deep aquifers of any significance, possibly excluding buried Karroo rocks (notably the volcanics) in the Lower Shire and also the deeper alluvium in the Lower Shire. The shallow aquifers behave in either unconfined or leaky artesian fashion. The main aquifers are not very permeable and consequently are not very productive. Possible exceptions may occur locally in the alluvial sequences but information is lacking. Water quality is mainly good to fair except in locations in the Lower Shire. Water quality in the deep alluvial sequences could well be saline.

6.2 Groundwater occurrence can most conveniently be considered in relation to the three main physiographic sub-divisions.

6.2.1 Alluvial deposits of the Rift Valley. These include sand, silts, clays and occasional gravels. The vertical profile is variable with a tendency for the deposits to become finer grained down gradient from source, i.e. towards the lake shore or Shire river. Dissolved solids in the ground waters increase in amount down gradient and may locally become excessive where recharge rates are low (notably in the Shire valley), or other contributory conditions occur.

6.2.2 The Rift Scarp Feature. On the dissected scarp, soil cover and other weathering products are thin and aquifers occur within the thin weathering sequence or fractured bedrock. Run-off is high on the steep slopes and recharge in consequence of this feature and the poor infiltration capacity of the thin soil and rock outcrops is low. Aquifers tend therefore to be poor and discontinuous.

6.2.3 On the Highland Plateaux, aquifers occur within the thicker weathering products and the fractured bedrock. The weathered zone may exceed 100 feet in thickness. Basement rocks are commonly isoclinally folded with steep dips. Zones of relatively high permeability forming the more productive aquifer units appear to occur with elongated form, often severll miles in extent. The feature may relate to bedded sequences within bedrock formations, e.g. quartzites, or to elongated fracture zones. There is a distinct vertical sequence with graded horizons as follows :

Laterite soils - several metres

2. Clays more or less sandy, of variable thickness and containing seasonal ground water.

3. Coarse sands and clays from decomposition of bedrock. Clays decrease with depth. This formation commonly constitutes the main aquifer with a piezometric surface which is frequently? higher in elevation than the phreatic water table.

Fractured bedrock.

RECHARGE

7.1 No quantitative assessments of recharge to the groundwater systems have been made in any of the more detailed local studies carried out to date.

7.2 Present groundwater abstraction is said (Wilderspin, 1973) not to exceed recharge. The evidence for this statement is not presented but is likely to be based on the moderate to good rainfall and the low density of boreholes which do not statistically exceed 30 in an 85 sq km rectangle. On an assumption of practical abstraction of hand operated equipment of 500 gallons per hour and a 12 hour day, total annual abstraction averaged over the 85 sq km would represent about 1/3 of a millimetre or less than 0.1% of total rainfall. The calculation presumes hydraulic continuity of the aquifer(s) below the area (the geochemical similarity of the groundwaters over much of Malawi (particularly in basement areas) is sometimes cited as evidence of the condition). On general principles there seems likely to be a more or less continuous weathered layer below the broad plateaux regions but there is also likely to be significant lateral variability in hydraulic and storage characteristics which will affect local well responses, and evidence for such occurrences have been quoted (Navarro, 1975). However, present total abstraction seems quite low and it seems a reasonable assumption that it is unlikely to be exceeding recharge except perhaps locally. Confirmatory evidence could most easily be obtained from a study of the trends in well water levels plotted from the borehole maintenance records. These trends have not been analysed to date although data from a selection of wells in one area obtained during this recent visit did show indications of falling levels (see section 9.3). To summarise, although current abstraction seems unlikely to be exceeding recharge, except locally, the heterogeneities within the aquifers will present problems in any overall resource evaluations.

7.3 Studies to determine recharge could be carried out along the following lines :-

General Assessments

(1) Rainfall - Runoff - Evapotranspiration (water balance studies)

(2) Seasonal water level changes. These figures can be obtained from the records of the borehole maintenance teams but consideration might be given to setting up of a well hydrometric network. A hydrometric network should initially be concentrated in areas of main exploitation and perhaps more particularly so in the areas floored by alluvial sequences where general hydraulic continuity within the aquifers probably exists.

(3) Base flow analysis in perennial stream systems. This study has been proposed by the Water Resource Division using past records of data which have been computerised to facilitate analysis.

(B) Detailed Assessments.

Studies in areas where more accurate assessments are required would need to determine a more precise water balance with consideration given to quantifying all the major parameters - abstraction, groundwater flow etc.

8. GROUND WATER ABSTRACTION

8.1 Hand Dug Wells.

Hand dug wells are the traditional mode of groundwater abstraction where the water table is at accessible levels. A programme is being promoted to construct hand dug wells fitted with small hand pumps and a covering cement apron which seals the surround and prevents any marginal leakage. The hand pumps are intended to be serviced by the beneficiaries and spare parts will be supplied. The total cost of materials is small (Kwacha 50 - verb. comm.).

8.1.1 The wells are of the order of 4-6 feet in diameter at the surface and reduce at a lower level. In practice only the upper section of some 2/3 feet is concrete lined and below this level the hole is generally open. Total depth is not likely to exceed 30 feet and most wells will probably be less. The common location for such wells is in dambos which are shallow depressions or valleys which are underlain by clays with sandy clay material above. The depressions are flooded during the wet season and water is retained in the sandier formations above the clays. It is estimated that such wells can produce up to 600 gpd and a density of 100 wells in 50 sq km is planned. Total annual abstraction of this order would represent the equivalent depth of less than a mm over this area. However, dambos occur over a limited portion of any area which will affect this correlation and many dambos are probably 'closed' systems.

8.1.2 The main problems in such wells are the limitations in storage and permeability in the dambo groundwater reservoir and pollution. Sites are generally located adjacent but not too close to a village and the well design is intended to provide additional protection. Cattle do however graze and water in the dambo areas and the possibility of pollution cannot be wholly discounted. Bacteria and virus travel distances and times vary in accordance with a number of factors, notably the fluid/air saturation of the medium, grain size and clay content. Shallow and fluctuating water tables as occur here constitute a particular hazard as would also conditions in which the hand dug wells penetrated into fractured bedrock (as seen in one well visited).. The two diagrams illustrated here give some idea of the uncertainties attached to pollution travel and 'safe distance' and it would seem adviable to initiate a programme of monitoring of a typical range of such wells in order to ascertain the effectiveness of the design in containing pollution. A further possible cause for concern could be nitrate (NO_3) concentration in the shallow groundwaters and the development could be emphasised in a 'closed' dambo system with no seasonal <u>through</u> flushing. The periodic recharge would be mixed with a residual storage in which nitrate concentrations might be continuously increasing. A programme of nitrate monitoring would be easy to initiate and is to be recommended.

8.2 Drilled Boreholes.

There are more than 3500 boreholes currently in operation, the majority being fitted with hand pumps and the few remainder with motor pumps. In view of the concern expressed by Government over the cost of constructing and maintaining boreholes, a careful appraisal of drilling techniques, well design, well yields and deterioration etc., is called for with a view to optimising procedures to reduce costs to a minimum and to make feasible proper cost comparisons with alternative sources of supply.

8.2.1 Boreholes are being drilled at a rate of some 300 a year in a general depth range of 130 to 200 feet. Percussion techniques are exclusively employed and 19 rigs are currently in operation (12 owned by private contractors, 7 by Government). The Government assumes responsibility for actual siting and supplies material and installs producting pumping equipment. As a general rule, a rig can drill and complete a well in about two weeks at an average (1976) cost of 2500 Kwachas(£1600).

8.2.2 The average yield during testing of the 306 successful wells drilled in 1976 was 867 gph. (Annual Report of the Geological Survey) With hand pumping equipment and under favourable circumstances with moderate draw-downs, such wells can yield between 300 and 500 gallons per hour. Normal production rates are thus less than estimated sustained yield and consideration could therefore be given to completing wells more cheaply and yet able to satisfy the requirements of hand pumping abstraction. In caution, it should be pointed out that the estimated sustained yields are based on 8 hour pumping tests and the quoted rates might well prove exaggerated in actual practice. If ever it is planned to use boreholes for maximum sustained production, much more comprehensive testing would be required.

8.2.3. Drilling techniques and construction design are highly standardised. Wells are commonly completed with 6 inch casing, plain or slotted, which is usually landed at total depth irrespective of the formation stability. The production casing programme is determined by the driller and usually consists of a 20 foot length of plain casing at the base to act as a sump succeeded above by 40 feet of torch or saw slotted casing (¼ inch slots). Temporary 8 inch or (rarely) 10 inch casing may be set in caving formations during drilling and subsequently withdrawn after installation of the 6 inch production string. At one of the contractors drilling sites visited during this current assignment, 6 inch 'drive' casing was being installed which was to serve eventually as production casing. Drilling was subsequently planned to continue below the driven section by open hole. Open hole completion is comparatively rare, even in relatively unweathered crystalline rock.

8.2.4 A gravel pack is emplaced in all wells whether completed in alluvial formations or weathered/fractured crystalline rocks. A standard volume of 3 cubic yards is utilised. Two grades of 'gravel' are used, a fine (⅛ - ¼") and a coarse grade (¼ - ½"). The material used is crushed crystalline rock (usually granulite) chippinga. Emplacement of the gravel is from the surface in the annulus between the open hole (which is generally in range 8-10 inch diameter) and the 6 inch production casing. If temporary casing has been used, it is withdrawn prior to pack installation. When the production casing has been driven, the same method is employed and emplacement is assisted by agitating the casing at the surface. Final completion of the well is effected by a natural clay filling in the top 10-20 feet of the annulus in order to prevent surface contaminants leaking downwards alongside the casing.

8.2.5 Well development is limited to surging with a 5½" diameter drill bit followed by pumping. When the discharge runs clear (more or less) an 8 hour production test is carried out using the rig reciprocating pump. The observed specific capacity is used to calculate the maximum safe yield assuming the total allowable drawdown to be from the water table to the top of the main aquifer. In the case of weathered basement rocks, this latter level is not always well defined but represents the transition from an upper more clayey to lower sandier material. The 'yield' quoted in the record is that produced by the reciprocating pump at a more or less standard r.p.m. The results of yield tests can thus be used for general comparison purposes.

8.2.6 As noted above, the pumping equipment installed in the majority of wells is hand operated and is of the standard piston type of lift pump with rods. The pump setting depth is related in practice to total well depth, the main variation being to fit the double wheel Climax type where the well is deep and the pump setting below 150 (?) feet.

8.3 <u>Well Maintenance and Costs.</u>

8.3.1 Hand dug wells. Maintenance of open hand dug wells is usually a matter of cleaning and/or deepening during periods of low water table. In the case of the covered hand dug wells, maintenance might also require similar procedures which would be less convenient since it would require breaking open and recementing the well cover. Normal maintenance is expected to be limited to replacement of the pump washers and it is intended that this operation will be performed by the local beneficiaries. Maintenance costs are to be met by Government and are planned to be limited to the almost negligible sums to replace pump washers.

8.3.2 Drilled Wells. For a 76 Kwacha annual charge, The Geological Survey maintenance unit provides a guaranteed service arrangement. Wells are visited on a routine basis or on demand following breakdown. A maintenance team operates with a 5 ton truck fitted with a winch to enable pumps to be pulled for service or repair. There are currently nineteen maintenance teams to visit the 3800 scheduled boreholes and visits average 3/4 a year per borehole. Total costs of the service are not known to the writer but they undoubtably exceed the imposed service charges. Routine maintenance involves little more than oil and grease jobs and no analysis is available on the nature of the various repairs. From discussions with two maintenance teams, many visits are occasioned by minor repair requirements of the surface installations. Where the pump has to be lifted, it is usually in order to have the pump leather washers replaced. A major problem which is particularly evident in wells completed in alluvial formations relates to sand entry into the boreholes. The process will cause a reduction in yield and in severe cases can result in casing collapse due to formation shift although this effect does not appear to have been recorded. Sand pumping will also cause excessive wear of the pump washers.

9. DISCUSSION ON PROCEDURES AND TECHNIQUES RELATING TO BOREHOLE SITING, DRILLING AND MAINTENANCE

9.1 A detailed discussion of these aspects to include consideration of costs, must form part of the main assignment but a preliminary assessment is appropriate in this report.

9.2 <u>Borehole Siting</u>.

Boreholes are drilled on demand at sites selected within a short distance of the point of utilisation. To date there have been no developments in any area which have required detailed appraisals of aquifers for optimum well field design and controlled abstraction. Wells are mainly required for domestic village supplies, either existing, or new settlements associated with agricultural projects.

9.2.1 Site selection is based on a rapid appraisal of the local morphology from maps, aerial photographs (sometimes) and ground traverses (terrain and vegetation) combined with a local geophysical reconnaissance employing an electrical resistivity technique. The procedure is to cover the general area (1-2 sq km) by a series of constant separation traverses using an electrode separation of 100 feet and a line separation of c. 600/700 feet. The results are expressed in the form of a contoured resistivity map. Expanding depth probes are then carried out along and in the vicinity of significant 'low resistivity' zones with a maximum electrode separation of 250 feet. Interpretation of the E.D.P's. is based on fairly simple curve matching procedures and attempts to distinguish (within the saturated zone) weathered or fractured porous and permeable rocks from clays (very low resistivity) or from hard impervious rocks (high resistivity). Problems in interpretation must occur when the low resistivity zone which the survey is seeking to identify is masked by near surface very low resistivity clays such as exist below 'dambos'.

9.2.2 A survey team consists of a geologist, technical assistant, 6 labourers and two drivers with associated transport. In Malawi, 3 such teams are engaged more or less full time on such work plus one geologist engaged on largely administrative matters relating to the work. A very approximate costing for survey per borehole allowing for staff costs, overheads, transport costs and vehicle depreciation is some 250 Kwachas. In 1976, 432 surveys were carried out and 390 potential sites found. Of this latter number, 334 were drilled and 306 were successful (yield in excess of 150 gallons per hour).

9.2.3 There is said to have been a marked increase in the success rate of boreholes following upon the use of electrical resistivity methods. In view of the apparent success rate, currently at 98%, there is little inducement to consider the use of alternative techniques or modification of procedures. The situation would appear different however if 'success' was redefined. Boreholes are in general considered very costly for domestic village supply and hence the emphasis on the self-help hand dug well programme. Boreholes are indeed considered too costly or too uncertain for other projects, e.g. the Bwanje valley, and alternative surface resources are being given active consideration. A more precise correlation of a borehole in relation to cost and proposed utilisation is needed to determine whether successful or otherwise and it seems probable that in these terms, many boreholes might be considered unsuccessful. A comprehensive appraisal of drilled boreholes would seem to be required in an attempt to define their most appropriate use and to optimise surveying and drilling procedures in relation to such use. Such an appraisal will require some analysis of record data but the following points are set out for preliminary consideration.

(i) The present survey routine should be examined to see whether it can be carried out more efficiently and/or more cheaply. At present, the total geological staff assigned to hydrogeological work (4) are concerned almost wholly with well siting. There is little if any time available to observe and supervise drilling, correlate drillers lithological logs with the earlier survey's predicted sequence, analyse well records or carry out other hydrogeological studies. The present surveying procedure is very routine and it is possible that it could be carried out quite adequately in many cases by a trained technical assistant with the geological staff providing some confirmatory interpretation. This arrangement would allow the geologists to do work on other hydrogeological aspects.

(ii) The resistivity method seems the most appropriate one for well siting on the basement rocks of the high plateaux. Because of the heterogeneous nature of the basement rocks, extrapolation from one location to another is not generally feasible and surveying is probably necessary for new sites even in the same general area where other wells occur. Consideration might be given to the use of an electro-magnetic technique which can provide a simple, rapid means for mapping subsurface conductivity variations. EM waves are attenuated rapidly by conductive material and penetration may be limited by heavy surface clay material. Where penetration would be adequate, reconnaissance mapping can be carried out very quickly by one man. High conductive zones could be studied in more detail, if thought necessary, by resistivity depth probing or there might be sufficient background information from earlier site surveys in the same area to obviate this need. In either case there would be a saving in time and cost.

(iii) EM traversing might also be considered for use in other circumstances. It might, for example, be used to survey an area rapidly for hand dug well sites by obtaining some measure of relative thickness of the sand formations above the basal clays in 'dambos'. EM techniques could prove more applicable to sites selected in the dissected rift scarp regions where rocky outcrops are common or underlie thin surface cover. The resistivity method has to date not proved very responsive in this region.

(iv) The use of resistivity survey for well siting in thick alluvial sequences seems less justified and examination of existing survey and drilling data may indicate that there is little value to be gained by its application. Of greater value might be closer study of existing well data to elucidate sedimentological and hydrochemical trends. In an area with no existing wells, some reconnaissance resistivity work would be more justified. Resistivity methods might also have a particular application in identifying subsurface saline water within an aquifer such as in the Lower Shire area.

(v) In certain areas, well sites have proved difficult to locate by resistivity techniques and the use of other geophysical techniques would be worth considering. In basement areas, E.M. and seismic methods could be appropriate.

(vi) As noted above, village wells fitted with hand pumps cannot usually be utilised at rates the wells are capable of yielding. Consideration should be given to drilling shallower wells which are adequate for the type of pumping equipment to be used and yet cheaper by virtue of savings in drilling costs. Drilled boreholes of intermediate depths would be more costly than hand dug wells but would generally have the advantages of a more assured supply and less risk of pollution. If these two factors are an important consideration in any area, drilled wells would be favoured in comparison with hand dug wells. Wells would need to be of sufficient depth to allow for annual variations in water table levels and to have sufficient penetration to give adequate yield with a hand pump. More precise geophysical correlation with lithology and depth would help in-situ selection of such wells and bailer pump tests should be carried out periodically during drilling to ascertain well yields. A statistical study of well yields versus depth in Malawi is in any case required to assist in decisions on continuation of drilling. There is always a tendency to continue drilling in the hope of encountering permeable horizons whereas the chance of so doing decrease rapidly with depth, particularly in basement rocks.

9.2.4 A different role for the geologists engaged in hydrogeological work in Malawi is envisaged. The approach to siting wells must correlate more closely with the nature and occurrence of the aquifer, the proposed utilisation of the groundwater and the yields required. For siting of boreholes or hand dug wells with small yields required, relatively cheap and rapid methods of survey should be utilised wherever possible. Where larger supplies are required with motor pump installation, the economics will usually justify a more comprehensive survey. In such cases, all available techniques should be considered including appropriate geophysical methods, aerial photographs and imagery, structural and fabric analysis of the associated rocks etc. Social considerations may also require comprehensive survey in areas where alternative and readily available sources of supply do not exist.

9.3 <u>Well Drilling.</u>

An examination of the well drilling methods in Malawi has indicated various possible improvements.

9.3.1 Drilling Technique. Drilling at present is carried out exclusively by normal percussion methods. The method appears generally adequate in relation to the depths drilled and in most formations, even including the alluvial deposits which tend to be clayey and rather fine grained. The method does have problems when collapsing formations are encountered at several depths. The use of 6 inch drive casing which is also production casing (the method was being practiced at one of the contractors drill sites visited) is not to be recommended as the slots are likely to be clogged up with clay etc. In such circumstances, a temporary drive casing programme of larger size, (8 inch, 10 inch or as necessary larger) should be used. This is in fact the standard procedure available.

9.3.1.1 An air rotary/hammer rig with ancillary mud circulation facility has been recommended for purchase. This rig would have many advantages in use provided that the operating personnel are sufficiently skilled and that the rig can be adequately maintained. The writers only other reservation relates to the proposed size of compressor of 250 psi. This is very large and will have a large fuel consumption. The high pressure rating will certainly increase the drilling rate in air hammer work but it is perhaps questionable whether this time saving will offset other disadvantages. The rating is not essential in other respects for air hammer or for air rotary in view of the planned drilling depths (generally less than 200 feet, rarely if ever exceeding 300 feet) and the low permeability (and consequent yield and associated fluid level drawdowns during drilling) of the formations to be encountered. In the writers opinion a compressor of 150 psi would be adequate.

9.3.2 Casing. Casing used in Malawi water wells is standardised at 6⅝" and made of mild steel. Two considerations apply here, size and material. A diameter of 6⅝" is larger than necessary for the hand pumping equipment in village domestic supply wells and could be effectively reduced to 4/4½". Steel casing is heavy, costly and susceptible to corrosion. Its main advantage is rigidity. Two alternative materials which could be considered are (1) fibre glass (GRP) and (2) various grades of thermoplastic material. Both alternatives are light and generally corrosion resistant. GRP has the advantage of highest material strength and could be used in all water wells in Malawi. It has the disadvantage of high cost (almost the same as steel) and the need to drill a relatively large hole to accomodate the larger size collars. Plastic casing is significantly cheaper than both steel and PVC (See Appendix A) and can be manufactured with threaded flush-butt joints. They have a more limited temperature range than steel or GRP but this deficiency is unlikely to be a matter of concern in water wells in Malawi. Their main disadvantage is an appreciable lower strength than either steel or GRP. In general PVC can rarely be used in wells exceeding 100 metres. The tougher thermoplastics can commonly be used in wells to 150 metres and locally deeper.

It is recommended therefore that consideration be given to using smaller diameter casing for domestic supply wells and also to install cheaper thermoplastic materials. It will be important to consider the strength priorities in the latter's selection and to carry out test installations in a range of formations and depths (basement, alluvials, etc.) prior to a major investment in a large order. Some preliminary assessment of strength limits can be deduced from drawdown data and particular care is necessary during development and test pumping when hydrostatic differential stresses will be at a maximum. Requirements for material strength in relation to longitudinal stresses can be readily calculated for the proposed emplacement conditions but response to radial stresses is more difficult to relate owing to uncertainties of formation support. Collapse resistance is likely to be the limiting factor in the depth of emplacement of thermoplastics.

9.3.3 Screen. Standard screen used is torch-slotted 6⅝" steel casing which in most cases is installed along with plain casing to total depth in all wells irrespective of rock type. Screen setting is selected by the driller and is customarily 2 x 20 feet joints. A fuller involvement by a hydrogeologist in this and other aspects of well construction is to be recommended. Open hole completion should be considered in all rigid formations, in particular crystalline rock. Screens should be set opposite the most permeable of the collapsing formations. Selection of such horizons has to be determined in the main from lithological logs and it is most important to ensure that samples are taken correctly (at one drilling site visited, samples were being thoroughly washed and would therefore be unrepresentative) in order to allow proper interpretation either by the senior driller or geologist. A general study using down-hole geophysical logging techniques (resistivity, gamma-ray mainly) is recommended in which a selection of wells in representative locations would be utilised. The study would assist correlation of surface geophysical data with lithological log interpretation from samples. Where time and opportunity permits, logs could be run before final completion, particularly in wells in variable alluvial formations for which a relatively complex screen setting is needed.

9.3.3.1 A particular problem in many wells in Malawi is sand incursion. The most common occurrences are in wells in fine-grained alluvial deposits but the condition is also known to occur in wells in weathered basement material. The condition results from an imbalance between the formation grain size and the gravel pack/screen slot size of the constructed well. Torch slotting cannot be cut smaller than 5 mm which is likely to be excessive opposite fine to medium grained sands unless a suitable gravel pack is emplaced. Drilling a large enough hole to accomodate a suitable gravel pack (recommended pack thickness 4-5 inches) will increase drilling costs and it is likely to be more economic in most cases to use a machine slotted screen for which smaller slot sizes are more feasible. The size of slot to be used will need to relate to the formation size distribution and cannot be determined otherwise. Another imortant consideration is the open area percentage. The screen currently in use has an open area of little more than 1% which must restrict fluid entry but increase inflow velocity and hence promote sand incursion. Mechanically slotted screens may have open areas in the 15-20% range which is hydrologically advantageous in several respects. The advantages of properly selected screens will become increasingly apparent when wells are constructed to obtain maximum abstraction at minimum cost in drilling and maintenance, e.g. shallow, small diameter wells for village supplies; deep, large diameter, high yielding wells for use with motor pumps. The cost implications will need to be recognised but in the writer's opinion, the cost benefit of production/maintenance are likely to outweigh the high initial cost of material. Drilling costs might also be reduced, if for example, no gravel pack needs to be emplaced by virtue of a suitable screen which can allow natural development of the aquifer. The use of a cheap thermoplastic screen is also worth considering as well as casing. The only uncertainty in the use of some thermoplastics is a tendency to 'creep' and in a screen this effect will cause the slots to close up. The better quality thermoplastics are likely to be less affected in this way, particularly when used in relatively shallow wells.

9.3.4 Gravel Pack. The type of gravel pack used in Malawi is too thin (about one inch), the gravel is generally unsuitable because of the limited size ranges (⅛ - ¼"; ¼ - ½"), and because the material is angular. The method of emplacement whereby gravel is fed-in from the surface between the production casing (6⅝") and the open hole (8"-11" or more) after withdrawal of any temporary drive casing is almost certain to be unsuccessful either by virtue of bridging of the gravel or by collapse of the surrounding formation into the annular space prior to emplacement. The data on volumes of gravel used are no guide to the amount emplaced since the standard amount of 3 cu yards in all wells is clearly a nominal figure. When the well is constructed in fine-grained unconsolidated material, sand incursion is almost bound to occur whether by virtue of an imbalance with a gravel pack too thin and too coarse a grade or directly through the wide screen slots around which the gravel has not extended.

9.3.4.1 A decision on the emplacement and type of a gravel pack should be based on standard hydrogeological principles with appropriate consideration being given to the type of formation and the screen slot size. In strongly or moderately consolidated rock types, neither a gravel pack, nor indeed in many cases a screen needs to be used. Where unconsolidated formations are encountered, a screen and/or gravel pack should be emplaced in accordance with the size distribution of the formation and to a limited extent with the well production rate affecting inflow velocity. There are advantages in dispensing with a pack if an appropriate slot size can be used and the aquifer developed naturally. Because of the manifest inefficiency of the gravel packs used in Malawi, it can be concluded fairly certainly that if wells in any area do not produce sand, wells of similar depth and yield can be constructed without a pack even with torch slotted casing. When sand incursion is common, representative samples must be collected to assist well design. A gravel pack of 1 inch thickness is theoretically adequate if it is of appropriate design (pack-aquifer ratio generally less than 5, and similar uniformity coefficients) but in practice it is not possible to install properly in a narrow annulus to any appreciable depth. If a gravel pack has to be used, a large size hole should be drilled and there would be advantages in using small diameter casing, e.g. 4 inch casing in a 10/12 inch hole. The gravel should be appropriately graded and composed of rounded, siliceous material. The higher cost of drilling is likely to be more than offset by reduction in maintenance costs and improvement in sustained production rates.

9.4 Well Development and Testing.

9.4.1 Well development is limited to surging with a 5¼" drill bit inside 6⅝" casing and pump testing for several hours using the rig reciprocating pump. Drawdowns are observed and a specific capacity calculated. Maximum yield is computed from the specific capacity and the available drawdown estimated as the distance between the water table and the top of the main aquifer deduced from the lithological log. This yield could not be regarded as a definitive sustained yield on the basis of such limited testing.

9.4.2 The method of surging is not very effective and a standard surge block would be better. Surging would need to alternate with bailing unless air lift pumping could be carried out simultaneously but suitable equipment is not generally available in Malawi. A close fitting surge block would be difficult to use as the metal 'swedge' left by the torch cutting operation would tear the rubber washers. A surge block could be used efficiently in a mechanically slotted screen which is another reason supporting the latters use. If proper screens are to be utilised both surging and jetting techniques would be appropriate for development.

9.4.3 Short duration bailer pumping tests are to be recommended during drilling in order to ascertain whether a sufficient depth has been drilled to provide the required production rate. The standard production test on completion is adequate in the present circumstances but would not be so if maximum sustained abstraction rates were planned.

9.5 Borehole Maintenance.

Two aspects of this work require mention. One relates to the periodic well water level observations and the other to the actual maintenance work required to be carried out on a well. Wells are visited on average 3/4 times a year partly in relation to a routine inspection which will be carried out irrespective of demand and partly in relation to requirement following reported breakdown.

9.5.1 Water level observations are made whenever a well has to be pulled. This data can provide important indication of the behaviour of the aquifer. In Table 1 are shown data on a number of boreholes in the North West Mzimba area with the corresponding rest water levels at the dates the wells were drilled and recent (1977-78?) values of water levels. Three wells show a small rise, 9 show water level falls, in some cases very substantial falls. These latter falls in level, if correct, would be a matter of some concern being indicative of at least locally depleting systems. There are probably other records in the files for these same wells and a plot of the well water levels with time will give a truer indication of actual trends.

9.5.2 The information on changing well water levels with time is of great importance. Past data can be analysed and will provide indications of spurious records, inequilibrated levels, etc. and hopefully genuine trends will also be recognisable. For the future it is essential to organise the collecting of such data to ensure that it is accurate. Each team should be provided with adequate water well level indicators which are easy to read and repair. A datum point should be established on a well (probably top of casing) from which all future records should be read. If for some reason, casing has to be cut during a repair, the new datum and its relation to the old datum should be noted. The water level should be recorded after the well has reached equilibrium. The time taken to pull a pump for repair is several hours, perhaps a day. Water levels should be taken prior to re-installing the pump and 2/3 measurements made towards the end of the repair job should indicate whether equilibrium has been reached. If the well has been out of action for some time, then equilibrium can be assumed.

9.5.3 Additional data (see below) is also recommended to be collected during routine well visits. The time taken to acquire the additional data will be negligible and the information obtained will be valuable.

(i) Pumped samples for electrical conductance measurements should be collected in areas where significant salinity variations are common e.g. the Lower Shire.

(ii) Plumbing the well by means of a reel, line and plumb bob. This would be done whenever a pump is pulled.

(iii) After re-installation of a hand pump, measurement of the time to fill a 4 gallon drum.

9.5.4 The proper recording of details of repair will assist in future planning to increase efficiency and cut costs. Some data is available and should be analysed with this objective in view. Maintenance is necessary but current costs are a matter of concern. Some aspects of maintenance in existing wells, e.g. sand incursion have to be accepted since there are no economic means to prevent its occurring (in the existing wells). Future wells in areas where the problem exists should be so constructed as to prevent ingress. It may be economically preferable to abandon a bad well requiring constant repair and to drill a new and better designed one. Certain other aspects of maintenance might be modified for improved efficiency. Simple well head repair and routine grease jobs could be carried out by one man on a motor cycle. The visit of the 5 ton lorry with winch, 6-man crew etc. should only occur when there is actual need to pull a pump, and perhaps bail out a filled well.

10. REGIONAL HYDROGEOLOGICAL/RESOURCE STUDIES AND PLANNING REQUIREMENTS

10.1 The discussions so far in this Report have been primarily concerned with methodology - of borehole siting, drilling and maintenance. For longer term planning, additional data are required to allow estimates to be made of resource availability and local development costs. Some areas in Malawi have been extensively drilled to exploit groundwater, principally in rural areas, for agricultural development, and detailed accounts of these areas are available in a series of unpublished reports of the Geological Survey (see review by Wilderspin, 1973).

The reports mainly discuss the hydrogeology and drilling aspects but rarely provide estimates of resource availability. In no cases are development costs considered. These studies should be reviewed and supplemented by data from borehole records in order to provide preliminary estimates of available resources. Where priority needs indicate,further studies to refine such estimates could be carried out. A more detailed study of one of these areas, the Lower Shire, is currently in progress and will continue for several (? 2/3) years under finance from the World Bank. The studies include all aspects of water use and resources in relation to agricultural developments. Other priority areas where water resource consideration will be required in relation to planned or current agricultural projects include the Upper Bwanje Valley, the Lilongwe region, and the West Mzimba Region.

10.2 Studies should commence immediately to provide the background data for the National Water Resources Plan. What is required is a preliminary appraisal of the overall water resources of Malawi in relation to convenient geographical sub-divisions and in terms of general availability (ground and surface supplies), order of development costs and usage. The following considerations would apply :-

(i) Areas in which groundwater is likely to be, in practical terms, the only resource available. The North West Mzimba Region probably falls into this category and perhaps even the Central Lilongwe Region. The study and development of groundwater in such areas could proceed along lines dictated by priority considerations, finance and available expertise. Preliminary assessments based on regional data (rainfall-runoff-evaporation) and localised information from existing well records should be made and would assist decisions on phased developments and/or further studies. Type of development would include consideration of shallow hand dug wells, and bored wells with hand or motor pumps. The role of the bored well and the hand-dug well needs careful consideration in relation to immediate and longer term requirements. The significantly cheaper cost of the dug well is a major advantage and may also permit more wells in any one area which is an added convenience. Problems of sustained supply and pollution are likely to be critical limiting factors. In certain circumstances, both dug and bored wells might be required to complement each other, with the latter providing supply during dry periods or eventually providing more convenient piped supplies to important focal locations from higher production wells fitted with motor pumps. Careful planning will be required to determine the most appropriate combination.

(ii) Areas in which surface supplies have a clear priority, e.g. foothill regions of Mlanje, Zomba and Nyika Plateaux.

(iii) Areas in which both surface and groundwater supplies could be considered, either conjunctively or alternatively and for which information on relative availability and comparative costs of development are necessary to assist decision making.

(iv) Water Use. Potential use includes domestic supply, industrial and irrigation use. The use of water for irrigation has perhaps not been given sufficient consideration due probably to the inadequate information available on potential reserves of groundwater and capacity rates. Areas such as the east bank of the Lower Shire where the alluvial formations may be high yielding could have particular potential in this role.

11. GENERAL CONCLUSIONS AND RECOMMENDATIONS

11.1 The report has considered briefly various possible modifications to procedures and techniques related to borehole siting, drilling and maintenance and these matters can be considered in greater detail during the forthcoming consultant assignment.

11.2 An overall appraisal of the groundwater resources of Malawi is needed which will delimit areas where such resources represent the only potential supply and others where both ground and surface supplies can be considered conjunctively and alternatively. Broad preliminary assessments of groundwater resource availability and development costings for such areas should be made on the basis of regional data on hydrology/meteorology combined with localised information from water well records. In consultation with Government in order to determine priorities, areas in which detailed studies are required will be recognised and suitable programmes of work proposed.

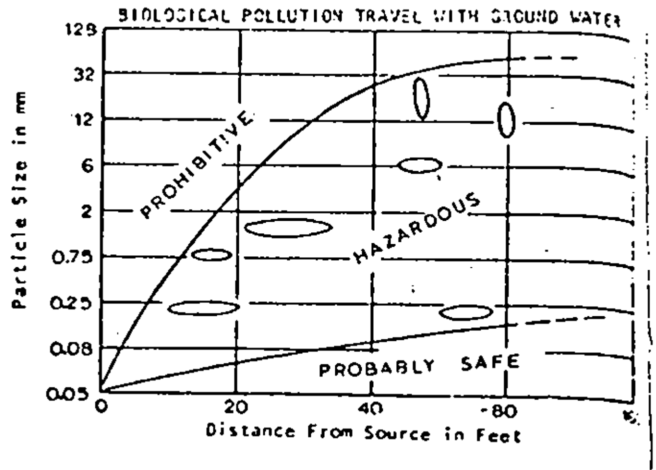
11.3 The role of the (4) Survey geologists associated with hydrogeology is at present almost exclusively concerned with well siting. Preliminary site surveys are essential in most cases but since much of the procedure is routine, it is suggested that such work be carried out in large part by trained technicians. The procedure would allow the geologists time to carry out other work - to supervise drilling and advise on appropriate well completion, assemble and analyse existing data from existing wells, maintenance and survey records, and to carry out some experimental studies on other geophysical techniques.

11.4 There is an obvious lack of expertise available in Malawi on hydrogeological matters. The four geologists assigned to the work are keen and enthusiastic but have had no formal training in hydrogeology. In the long term this could be remedied by sending suitable candidates to undertake post graduate training which would last at least one year to be effective. There is however a considerable urgency in the present situation to provide quantitative data for areas on which important development decisions will be taken in the near future. It cannot be anticipated that the 3 month assignment will supply this deficiency and it is certain that whatever detailed studies are proposed, substantial outside assistance will be necessary if such studies are to be carried out in reasonable time. The form of the desired assistance will be a matter for consideration by the Government of Malawi. However from the point of view of efficiency and time, there would be some merit in perhaps renewing or extending the present planned short term assignment to an arrangement of longer duration. There is a considerable backlog of record data to assemble and process and training of counterpart staff could also be commenced. The procedure would not preclude other forms of input on selected studies.

E. P. Wright
June 30th 1978

REFERENCES

	M.G. BEYER	Groundwater Production from the Bedrock of Sweden.
	I. LARSSON	Groundwater in Pre-Cambrian Rocks in Southern Sweden.
		Both from Proc. of Int. Sym. on Groundwater Problems, Stockholm, October 1966.
	G.S. CARTER and J.B. BENNETT	Geology and Mineral Resources of Malawi. Bull. 6M of Geological Survey of Malawi, 1973.
	K.E. WILDERSPIN	A General Account of the Groundwater Resources of Malawi. Report of the Geol. Surv. of Malawi, 1973.
	R.E. DIJON	Report on a Mission to Malawi. U.N. Report 1972
6.	A. NAVARRO	Report on a Mission to Malawi. U.N. Report 1975
	S.N. DAVIS and L.J. TORK	Optimum Depth of Wells in Crystalline Rocks. Groundwater, 1963.



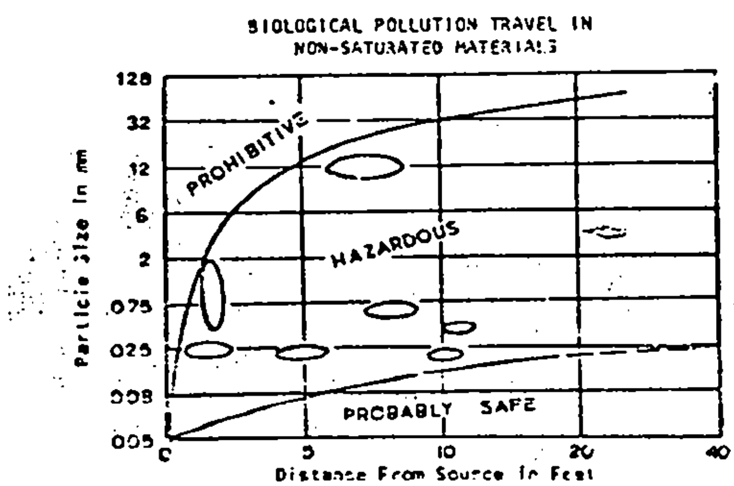


Fig. 2. Biological pollution travel in nonsaturated materials.

Appendix A. Costs Comparisons.

DESCRIPTION.	CASING COST.	COST OF CORROSION PROTECTION.	COST OF SHIPPING / HANDLING.	DRILLING / INSTALLATION SURCHARGE.	COST OF DAMAGE / BREAKAGE.	TOTAL INSTALLED COST.
Steel API 5A. Threaded / Coupled.	10	0.10	2.24	1.94	0.12	14.40
GRP.	10.8	-	1.54	1.50	0.24	14.08
uPVC.	4.44	-	1.28	-	0.57	6.29
PVC / ABS (Terraline 60)	4.06	-	1.28	-	0.27	5.61
ABS (Terraline 85)	6.39		1.28		0.15	7.82
Polyolefin. (Terraline 110)	5.54	-	1.28	-	0.14	6.96
HDPE.	5.24		1.28		0.13	6.65

TABLE 1

BOREHOLES IN NORTHWEST MZIMBA

G.S. No.	Depth, Feet	Yield G.P.H.	Water Level, Feet	Water Rest Level, Feet	Date Drilled
GK 244	235	200	97	77	1977
PM 21	140	510	102	100	
W 85	104	720	5	62	1962
X 168	153	600	55	63	1971
RB 82	145	540	20	33	1972
X 169	222	500	55	22 ? 122	1971
L 404	122	163	10	28	1957
Q 394	155	540	40	44	1970
Q 395	151	750	18	18	1970
X 163	127	580	45	70	1971
X 164	206	214	70	119	1971
X 170	141	300	50	22	1971
R 183	91	450	20	18	1970